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AND

DISCUSSIONS

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    * Publication of closing discussion pending.

PANAMA CANAL—THE SEA-LEVEL PROJECT

A SYMPOSIUM

NOTE.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by September 1, 1948.

THE FUTURE AND THE PANAMA CANAL

By James H. Stratton,[1] M. ASCE

Synopsis

Investigations under Public Law No. 280 (Seventy-ninth Congress, First Session, approved December 28, 1945) disclose that only an Isthmian sea-level canal will meet the future needs of interoceanic commerce and national defense. The broad phases of the studies leading to this conclusion are described in this paper. Various engineering features of the studies of the recommended sea-level canal are described more fully in the other papers of this Symposium.

The Canal Zone offers the most economic site for either a lock or a sea-level Isthmian canal. The present lock canal could be improved at a cost of \$129,983,000 to meet the needs of commerce for the remainder of the twentieth century. A lock canal designed to meet the future needs of commerce and constructed to have the maximum security feasible in this type of canal would require new locks and strengthened summit lake impounding dams. It would cost \$2,307,686,000 and would still be deficient in resistance to modern weapons.

A sea-level canal at Panama constructed by the conversion of the existing lock canal could not be destroyed by enemy attack or sabotage. Only the atomic bomb could cause significant interruption in service, and then for not more than a few weeks. Navigation would be practicable in the sea-level canal even though tidal currents were not regulated. Nevertheless tidal regulation would be provided for greater safety to shipping. For this purpose a tidal lock and a navigable pass would be provided. The opening of the gates of the navigable pass at selected Pacific tidal stages would permit the routine operation of the canal as an open waterway. In the event of damage to the tidal-regulating structures, the gates of the navigable pass could be removed quickly and the canal could be operated thereafter as an open waterway. Serious slides could be prevented by appropriate flattening of the slopes in cut. The entire excavation to effect the conversion to a sea-level canal would be completed in advance of conversion; the major part would be in the dry. Part of the wet excavation would require the use of special dredges working to a depth of 145 ft. Construction interference with canal traffic would be negligible except for a period of a few days when the summit lakes are lowered. The Panama sea-level canal would be constructed in 10 years and would cost \$2,483,000,000.

Introduction

The long history of the search for a passage to the east, and of the early efforts to promote a canal across the Isthmus of the Americas, provide a background of romance and color to the story of actual construction which opened

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in 1882 with the initiation of work on a sea-level canal at Panama by the first French Canal Company. After years of effort and mounting adversities when funds had run low, the French changed to the less costly high-level lock canal. Because the French believed that a sea-level canal ultimately would be necessary, plans for the locks were drawn to facilitate the transformation. The chapter of the story dealing with the French construction ended with financial failure and the transfer of all rights and interest in the Panama Canal to the United States.

The opening chapter of the tale of construction of the Panama Canal by the United States is one of controversy over the type of canal to be built. The records disclose that the selection of the lock canal was based on the advantages it offered in the earlier completion at a lesser cost.[2] The issues entering into the
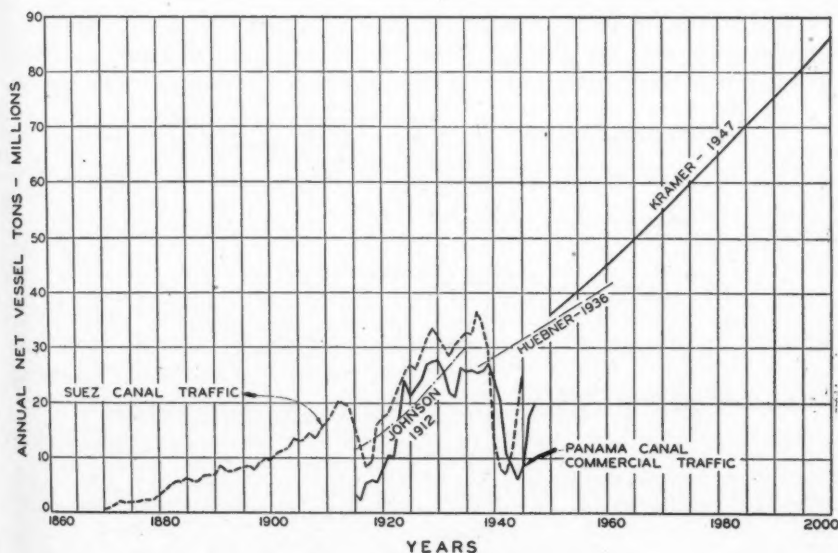
FIG. 1.—PANAMA CANAL TRAFFIC PREDICTIONS

selection have a particular interest now, in the face of the challenge presented by the "blockbuster" bomb, the guided missile, and the atomic bomb. When the present canal was planned the critical forms of attack were envisioned as naval gunfire directed against the locks and enemy forces moving overland to capture the canal intact.

The Seventy-ninth Congress expressed the temper of present concern for the security of the canal by passing Public Law No. 280 which was approved by President Harry S. Truman on December 28, 1945, and which provides:

"Be it enacted by the Senate and the House of Representatives of the United States of America in Congress assembled, That the Governor of the Panama Canal, under the supervision of the Secretary of War, is hereby

[2] "Report of the Board of Consulting Engineers for the Panama Canal, 1906," U. S. Govt. Printing Office, Washington, D. C., 1906.

authorized and directed to make a comprehensive review and study, with approximate estimates of costs, of the means for increasing the capacity and security of the Panama Canal to meet future needs of interoceanic commerce and national defense, including restudy of the construction of additional facilities for the Panama Canal authorized by the Act approved August 11, 1939 (53 Stat. 1409). He shall also make such study without drafting plans or sketches as he may deem desirable to permit him to determine whether a canal or canals at other locations, including consideration of any new means of transporting ships across land, may be more useful to meet the future needs of interoceanic commerce or national defense than can the present canal with improvements. He shall report thereon to the Congress, through the Secretary of War and the President, not later than December 31, 1947."

Traffic History of the Canal

The volume of traffic through the Panama Canal has steadily increased since it was opened, despite the setbacks of wars and world depressions. With uninterrupted world prosperity a further growth of commercial traffic may be
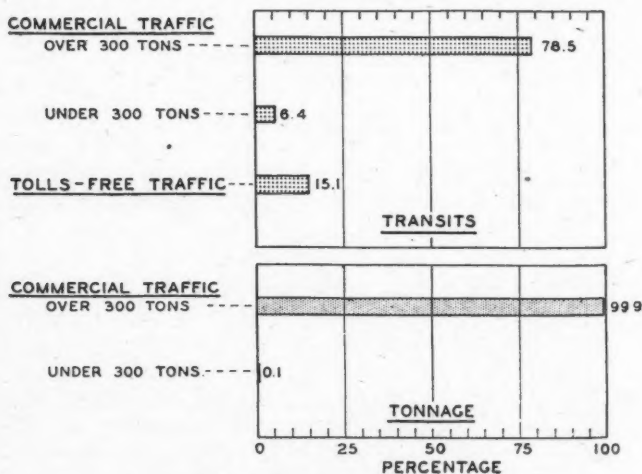
Fig. 2.—Predicted Daily Traffic

expected generally as predicted by Roland L. Kramer of the University of Pennsylvania at Philadelphia in recent studies which were made for use in the investigations under Public Law No. 280. Fig. 1 records past traffic and depicts the estimated and future commercial traffic, as projected by Professor Kramer.

Before the war the "tolls-free" traffic, consisting largely of vessels owned and operated by the United States, accounted for about 15% of the entire traffic. In 1945, the peak war traffic year, this rose to 78%. Predictions of future daily traffic are presented in Fig. 2. Since the canal was opened in 1914 it has transited 198,000 ships of which 142,000 were tolls-paying commercial craft. The percentage distribution of transits and tonnage is illustrated in Fig. 3.

War traffic through the canal in the period from 1941 to 1945 totaled nearly 17,000 transits. Had there been no canal during this period, it is estimated that the increased ship-operating costs and the cost of additional shipping and escort craft that would have been required to preserve the schedules made possible by the canal would have exceeded $1,500,000,000.
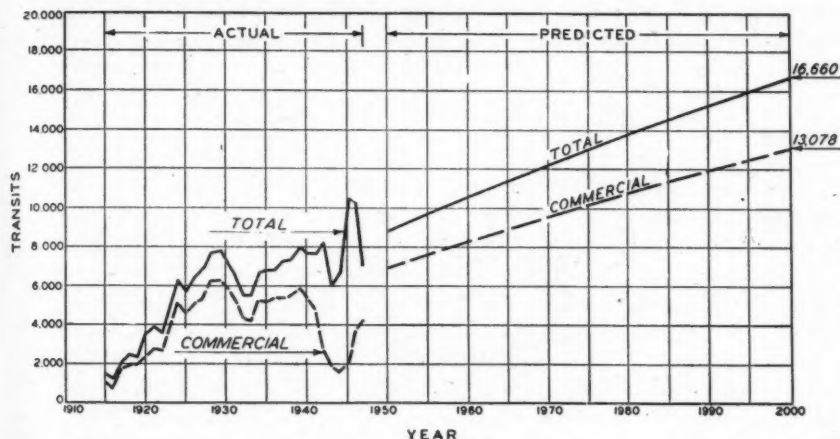
FIG. 3.—PANAMA CANAL TRANSITS

From its contribution in peace and war, it is clear that the future of the canal is the future of ships, provided the canal can be made safe against destruction. As Norman Padelford has stated:

"* * * the Panama Canal is more than a commercial artery. * * * It is a focal point of national defense, a base of operations for the protection of the Hemisphere, an instrument of national influence."

"* * * it seems safe to say that so long as any material part of the commodities of trade are carried in ships, and so long as sea power persists as a determining factor in the relationships of nations, so long certainly will use of the Panama Canal be sought by merchantmen and vessels of war. And so long will the Canal as a waterway remain essential to the United States. * * * No Panama Canal would exist today to pass great ships from ocean to ocean had it nor been for vision which saw beyond the limitations of existent realities. The hope for a more ordered future * * * lies in similarly transforming present difficulties through enlightened leadership and continued vision."[3]

THE PANAMA CANAL AS IT IS TODAY

The canal as a waterway has changed little since it was placed in service, except for the addition of the Madden Dam on the Chagres River in 1935 for the benefit of water supply for lockages, flood control, and power generation.

A plan and profile of the existing canal are shown in Figs. 4 and 5. It is estimated that by about 1960 the capacity of the canal will be inadequate to accommodate traffic without inflicting undesirable delays on peak traffic days.

[3] "The Panama Canal in Peace and War," by Norman J. Padelford, The Macmillan Co., New York, N. Y., 1942.

The delays thereafter will become more serious with the further growth of traffic unless additional capacity is provided.[4,5]

The restricting effect of the small locks (width 110 ft, length 1,000 ft) of the present canal on the design of Navy ships became intolerable with the approach of war and, in 1939, Congress directed the construction of a third set of locks, 140 ft wide and 1,200 ft long, which it was then thought would be adequate for all future needs. The new locks were designed to resist attack by
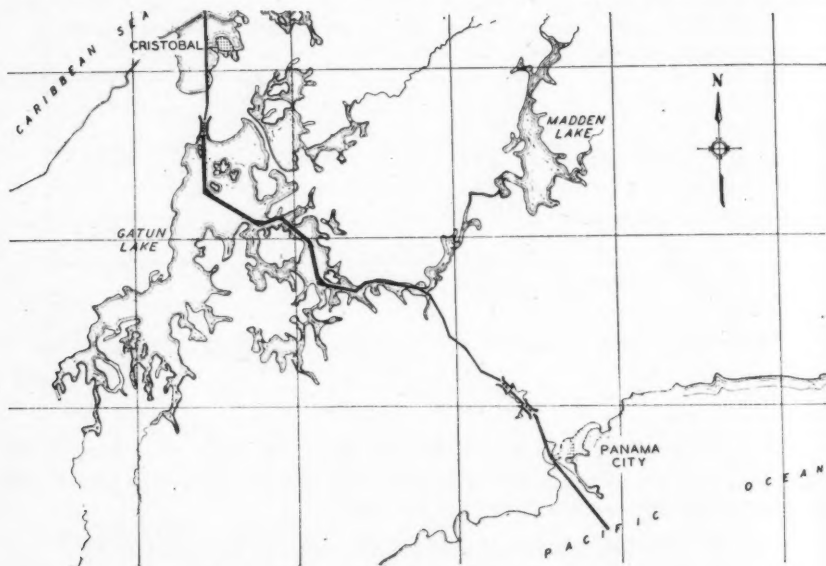
FIG. 4.—GENERAL PLAN, PANAMA LOCK CANAL

the largest aerial bomb then known to exist. Construction was suspended early in 1942, when it became apparent that the new locks could not be completed before the end of the war because of conflicting demands for men and materials. At that time, excavation for the Gatun and Miraflores Third Locks had been substantially completed, but excavation for the third lock at Pedro Miguel and work on actual lock construction had not commenced. Of the authorized expenditure of $277,000,000, approximately $75,250,000 was spent.

CAPACITY LIMITATIONS OF PRESENT CANAL

The present locks are expected to be adequate dimensionally for all commercial shipping for the remainder of the twentieth century except for ships of the "Queen" class which do not, and ordinarily would not, use the sea route through the canal. The limiting effect of lock size on the passage of naval ships is expected to become even more stringent in the future than it is at present.

[4] "The Isthmian Canal Situation," by Hans Kramer, *Transactions*, ASCE, Vol. 94, 1930, p. 406.
[5] "Additional Lock Facilities for Panama Canal, 1939," *House Report No. 494*, 76th Cong., 1st Session, 1939.

The individual locks of the present canal periodically are taken out of service a lane at a time for overhaul, thus the dependable or firm capacity of the canal is not that established by the twin lanes of locks but is that established by the single lane locks available in the extended periods of overhaul. Occasional night fogs require closing Gaillard Cut to traffic after midnight in the rainy season.   The locks are overhauled in the dry season when round-the-clock operation of the canal is feasible, thereby avoiding the reducing effects of both lock overhaul and fog on the canal capacity.

FUTURE TRAFFIC NEEDS

The commercial canal tonnage predictions of Professor Kramer were transformed into expected future ship transits by the engineering staff employed on the studies by taking into account the trends in ship sizes, the expected character of future cargoes, and the expected proportion of transits under full and partial load, and in ballast, as evidenced by past experience. Thus, the transportation of 87,770,000 long tons of commercial cargo estimated by Professor Kramer for the year 2000 would require 13,078 ship transits.   In the year 2000, commercial and tolls-free traffic would average 46 transits daily.   On peak traffic days 69 transits could be expected.   The year 2000 was selected to establish the period during which future traffic needs must be met if any major construction or reconstruction is undertaken.

By locking small ships in tandem and taking into account the sizes of ships estimated for the future, the peak load of 69 ships could be realized with 46 lockages using the present locks.   With locks 200 ft wide and 1,500 ft long (which is the size now recommended by the Navy to meet its future needs), it would take 29 lockages to effect the passage of the 69 ships.

HOW SECURE IS THE PRESENT CANAL?

Although no one can say what course World War II would have taken had the Japanese followed up Pearl Harbor (Hawaii) with an attack on the Panama Canal, it is now clear that the locks could have been destroyed and the canal emptied into the sea had an attack been made and the defenses penetrated. The development of larger bombs and new weapons of both conventional and atomic types since Pearl Harbor leaves no doubt as to the vulnerability of the canal to enemy attack and sabotage.   If the needs of national defense are to be met, the canal must be made secure against attack and sabotage.

New measures offering effective resistance to the penetration of defenses by rockets and guided missiles have not kept pace with the development of offensive weapons and indeed the prospects, for the future, of the defense are so gloomy it appears that the historic pattern of war may never be restored. The advantage now lies entirely with the aggressor who undertakes to destroy his enemy's ability to wage war by the wholesale destruction of his population and of his instruments of war before he can employ them.   Thus, to live as a nation the United States must shield its own vital weapons so that it can take countermeasures to prevent exploitation of any initial advantages the enemy may gain by surprise attack.   Prudence requires also that communications,
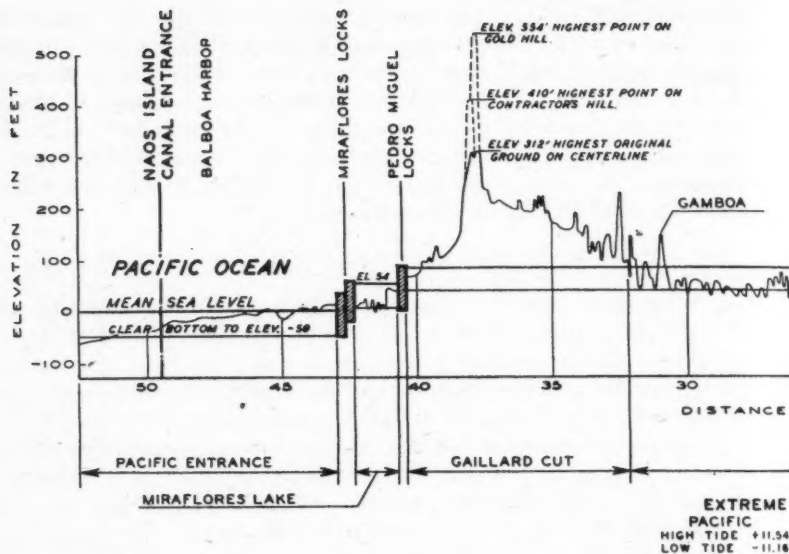
FIG. 5.—PROFILE,

such as the Panama Canal, and essential war industries be planned so that the nation would not be hopelessly crippled if attacked.

The penetration of canal defenses by rockets, guided missiles, or robot planes loaded with powerful conventional or atomic explosives launched from the air, from ship or submarine at sea, or from a land base, must be accepted as a possibility. A single sneak attack could destroy the lock gates of the present canal and drain Gatun Lake to the sea.

If the needs of national defense are to be met, steps must be taken to make the canal secure. No matter how resistant to attack the canal is made, however, an alert and modern defense is needed to warn the enemy that an attack will be costly in men and material, to prevent its capture, and to keep the enemy from having a free hand in sinking ships in transit and in making havoc of the auxiliary facilities of the canal.

THIRD LOCKS PROJECT IN REVIEW

A new third set of locks to accommodate large naval vessels, no matter how strongly constructed to resist attack and sabotage, would not add to the overall security of the canal because the existing locks and impounding dams cannot be strengthened sufficiently to give them equivalent protection. If any lock is breached and Miraflores Lake or Gatun Lake is lost, the canal would be closed for months or even years for repairs and for the restoration of the lost lake. In the light of the threat of present weapons, the Third Locks Project would provide for only the peacetime needs of commerce, which, if that were the sole consideration, could be met by other means at a considerably lesser cost.
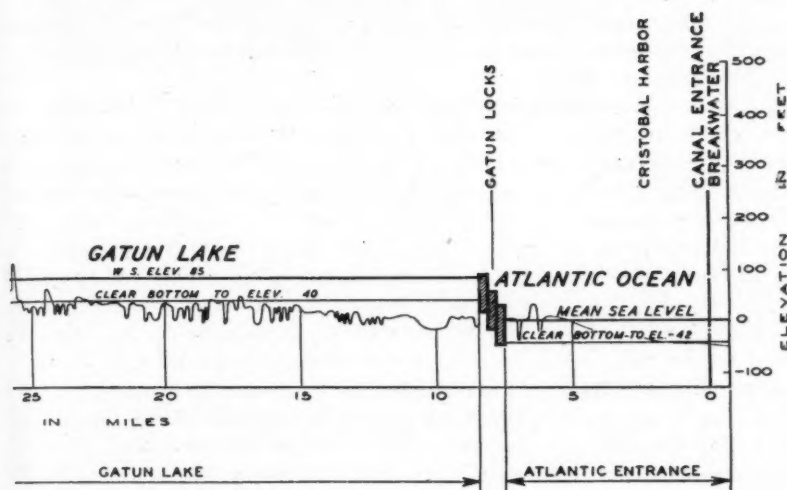
TIDE RANGES
  ATLANTIC
    HIGH TIDE + 1.8
    LOW TIDE - 1.25'

Panama Lock Canal

IMPROVEMENTS IN THE INTERESTS OF COMMERCE ONLY

The requirements of interoceanic commerce for the remainder of the twentieth century could be met by the elimination of lay-up of the locks for overhaul and by overcoming fog interference with traffic. The first of these is the more restrictive on the capacity of the canal, reducing it from 58 ships to 36 ships per day. Each lock is now overhauled every 4 years, repairs being undertaken every 2 years, alternating between the Atlantic (Gatun Locks) and the Pacific Locks (Miraflores Locks and Pedro Miguel Locks). While one lane of locks is under repair, the adjacent lane of locks is kept open to traffic; thus the availability of only a single operating lane of locks at one end or the other of the canal, during the period of overhaul (about 4 months), establishes the dependable canal capacity. Repairs are made in the dry season when there is no fog, and the canal is then operated 24 hours daily instead of 16 hours per day as is the case in the rainy season when fogs are of relatively frequent occurrence after midnight. Round-the-clock operation and careful scheduling of transits make it possible to hold the reduction in capacity due to overhaul of the locks to less than one half of the normal operating capacity of the canal.

Repairs to the lock gates and their mountings, and to the operating machinery and the filling culvert valves and fittings, are undertaken with the lock chamber dewatered, as are the cleaning and painting of all underwater metal parts. The lay-up and dewatering of the locks for repairs could be eliminated by providing new gate mountings of a special type and new type lock gates having buoyancy chambers to float them out and into position, thus effecting replacements in the matter of a few hours. Repairs to gates would

be made in dry dock.   Alterations to avoid lay-up of the locks for overhaul and certain channel improvements would raise the dependable capacity of the canal to 65 ships per day.

Fogs of the ground-radiation type which occur in Gaillard Cut in the rainy season (generally from May through December between midnight and day-break) require closing the canal to traffic during these hours.   Various methods of dispersing fog have been employed for the clearing of airfields, but present indications are that they would be too expensive for dispersing fogs in the canal.   Developments in electronic aids to navigation offer the prospect of a cheap and fully reliable method of passing ships through a fogbound restricted channel.   By the time traffic demands require 24-hour operation of the canal, these devices will be considerably improved and undoubtedly can then be adapted to the canal needs to increase its dependable capacity to 70 ships, which would be adequate for the remainder of the twentieth century.

The cost of the various improvements to extend the life of the present canal in the interests of commerce only would be $129,983,000.   Any expenditure in excess of this amount can be justified only by the requirements of national defense.

A New Lock Canal For Security

Although it became apparent early in the studies that an increase in security to meet the needs of national defense could not be attained through the rein-forcement of the locks of the present canal, this conclusion of itself did not eliminate the possibility of its attainment by the complete reconstruction of the existing lock canal or by the construction of a new lock canal elsewhere.

A survey of available reports of previous explorations and investigations and the current studies revealed thirty lock canal routes, of which many are merely alternate alinements of well-known routes:[6]

Route	Description
1........	Tehuantepec
	Nicaragua, via Lake Nicaragua—
2........	Greytown-Fonseca Bay ⎫
3........	Greytown-Realejo ⎬ Via Lake Managua
4........	Greytown-Tamarindo ⎭
5........	Greytown-Brito
6........	Greytown-San Juan del Sur
7........	Greytown-Salinas Bay
	Nicaragua—
8........	Greytown-Salinas Bay
	Panama—
9........	Chiriqui
10........	Chorrera-Lagarto ⎫
11........	Chorrera-Limon Bay ⎬ Alternate sea-level routes, Canal Zone and
12........	Chorrera-Gatun ⎭ vicinity

[6] "U. S. Army Interoceanic Canal Report on Panama Canal and Nicaragua Canal, Sultan Report, 1932," *House Document No. 139*, 72d Cong., 1st Session, 1932.

Route	Description
13........	Panama parallel
14........	Panama sea-level conversion
15........	Panama Canal, route of the present lock canal
16........	San Blas
17........	Sasardi-Morti
18........	Aglaseniqua-Asnati } Caledonia Bay routes
19........	Caledonia-Sucubti

Panama and Columbia; Tuyra River Routes—

20........	Tupisa-Tiati-Acanti
21........	Arquia-Paya-Tuyra
22........	Tanela-Pucro-Tuyra
23........	Atrato-Cacarica-Tuyra
24........	Atrato-Peranchita-Tuyra

Colombia; Atrato River Routes—

25........	Atrato-Truando
26........	Atrato-Napipi
27........	Atrato-Napipi-Doguado
28........	Atrato-Bojaya
29........	Atrato-Baudo
30........	Atrato-San Juan

Excavation estimates based on existing maps, supplemented in some cases by field and air reconnaissance, resulted in narrowing the choice of routes for a lock canal to the eight listed in Table 1 and shown in Fig. 6.

TABLE 1.—COMPARISON OF LOCK CANAL ROUTES

ROUTE[a]		Length (miles)	Elevation of divide at crossing[b]	Channel excavation[c] (million cu yd)	Elevation of summit lake[b]	Lock lifts per lane[d]	Approximate cost (million dollars)	Action
No.	Place							
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	Tehuantepec	165	812	3,360	550	10	13,280	Eliminated because of higher cost.[e]
5	Nicaragua..	173	153	1,060	110	2	3,566	Eliminated on the same basis as route 1.[e]
9	Chiriqui....	55	5,000	....[f]	....[f]	....[f]	....[f]	Eliminated because of height of divide.[g]
15	Panama....	51	340	191	92	2	2,308	Retained for final study.
16	San Blas....	40	1,100	1,480	110	2	5,960	Eliminated because of excessive cost.
19	Caledonia...	63	1,100	1,110	110	2	4,751	Eliminated because of excessive cost.
23	Tuyra River	135	470	1,120	110	2	....[h]	Eliminated because of excessive cost.[g]
25	Atrato River	95	932	1,450	177	3	....[h]	Eliminated because of excessive cost.[g]

[a] See Fig. 6.   [b] In feet above mean sea level.   [c] Channels are 500 ft wide at a depth of 40 ft and 55 ft deep below low water level.   [d] Two lanes of locks in each route, the lock chambers being 200 ft wide, 1,500 ft long, and 50 ft deep.   [e] After full consideration of shipping benefits from shortening trade routes. [f] Details not developed.   [g] Excessive excavation.   [h] Not estimated.

None of the seven lock canal routes at locations remote from Panama offers compensating advantages in additional security or in savings through the shortening of trade routes over the Panama lock canal route.

The plan of development used in the comparative lock canal studies was the same in each case and is as described in the next section for a modernized lock canal at Panama having the maximum security feasible in this type of canal.
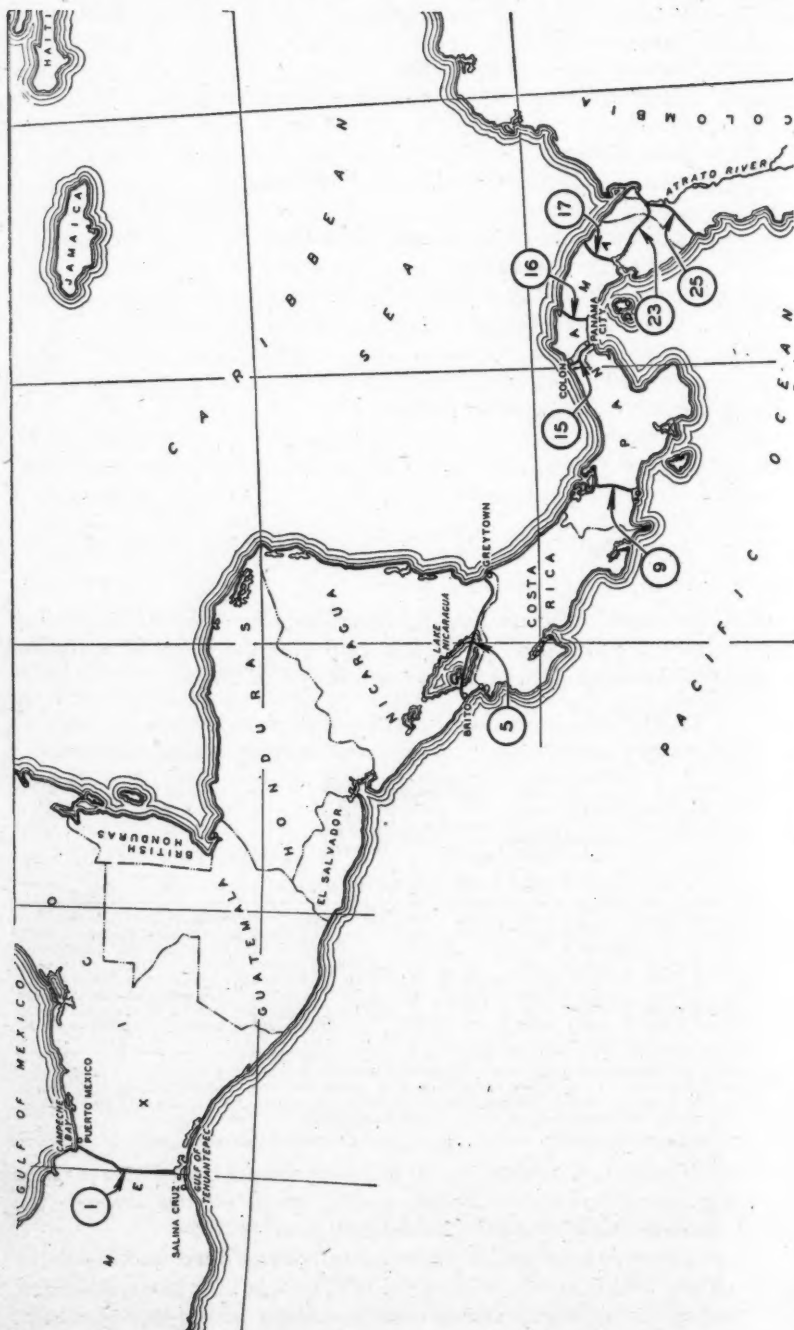
Fig. 8.—Transisthmian Canal Routes

A Modernized Panama Lock Canal

A reconstructed lock canal with all Pacific locks grouped at Miraflores to provide full lift to summit level would offer the greatest economy of construction and would create an anchorage area at the head of the Pacific locks that would facilitate the dispatch of vessels through Gaillard Cut. A similar arrangement of the Pacific locks was proposed by Adolphe Godín de Lépinay in 1879 and was advocated by the late W. L. Sibert, M. ASCE, in 1908. Colonel Sibert's suggestion was rejected because of the advanced state of the planning and construction, and doubts as to the foundations for three-lift locks at Miraflores, the site he proposed for the Pacific locks. The Miraflores foundations have since been found satisfactory for locks with full lift to summit level.

Two dispersed locks would be provided at Gatun and Miraflores with two lifts to attain summit level instead of three as at present. Chambers would be 200 ft wide, 1,500 ft long, and 50 ft over the sill. By raising Gatun Lake to El. 92, from its present maximum El. 87, additional storage for lockage water would be provided. This augmented water supply would not meet lockage demands until the year 2000 and supplementary pumping from the sea eventually would be necessary.

The dispersion of the locks and their armoring with concrete and steel to protect the lock machinery and the culverts would provide them with the highest practicable degree of protection. The lock gates do not lend themselves to protective treatment, except against the lightest type of aerial bomb. However, multiple sets of gates, well dispersed, would increase the difficulties of dissipating Gatun Lake. Certain of the gates would be housed in protected recesses when not in use. The closure dams adjoining the locks at Gatun and Miraflores would be of massive earth construction. The Gatun Dam spillway would be channeled in the rock abutment for maximum protection.

The cost of such a fully modernized lock canal at Panama would be $2,307,686,000.

Security considerations restrict public evaluation of the protective designs in relation to the various weapons that could be employed in an attack. It can be stated, however, that the lock canal cannot be made resistant either to atomic bombs or to modern conventional weapons. At best the protection that could be provided a lock canal would only increase the difficulties rendering it useless. The modernized locks could be breached by a determined enemy and the canal could thus be closed to traffic for the period required for reconstruction and for the capture of the tributary runoff to restore the summit lakes, which might require as much as 4 years. The extent of damage and the length of the period of traffic interruption would depend on the nature of the weapon employed and the intensity of attack. Radioactive contamination would make repairs to the locks difficult if not impossible. The lock type of canal, no matter how strongly constructed, would not increase security to meet the needs of national defense.

Sea-Level Canal Possibilities

The various sea-level canal route possibilities were narrowed down, in the same manner as were the lock canal routes, to the eight listed in Table 2, which

includes the principal features, quantities of excavation, and the estimates of
cost.   The Panama route (Fig. 7) is the least costly and has the additional
advantages of an operating and administrative establishment and of defenses

TABLE 2.—COMPARISON OF SEA-LEVEL CANAL ROUTES

Route[a]		Length (miles)	Elevation of divide at crossing[b]	Channel excavation[c] (million cu yd)	Approximate cost[d] (million dollars)	Action
No. (1)	Place (2)	(3)	(4)	(5)	(6)	
1	Tehuantepec....	165	812	6,130	....[e]	Eliminated because of excessive cost.[f]
5	Nicaragua......	168	760	5,200	....[e]	Eliminated because of excessive cost.[f]
9	Chiriqui.......	55	5,000	68,800	....[e]	Eliminated because of height of divide.[f]
15	Panama........	46	410	1,069	2,483	Retained for final study.
16	San Blas.......	40	1,100	2,080	6,272	Eliminated because of excessive cost.
19	Caledonia......	59	1,100	1,880	5,132	Eliminated because of excessive cost.
23	Tuyra River....	135	470	2,140	....[e]	Eliminated because of excessive cost.[f]
25	Atrato River ...	95	932	1,810	4,594	Eliminated because of excessive cost.

[a] See Fig. 6.   [b] In feet above mean sea level.   [c] Channels are 600 ft wide at a depth of 40 ft and 60 ft
deep below low water level.   [d] Exclusive of tidal regulating structures, except at Panama.   [e] Not esti-
mated.   [f] Excessive excavation.
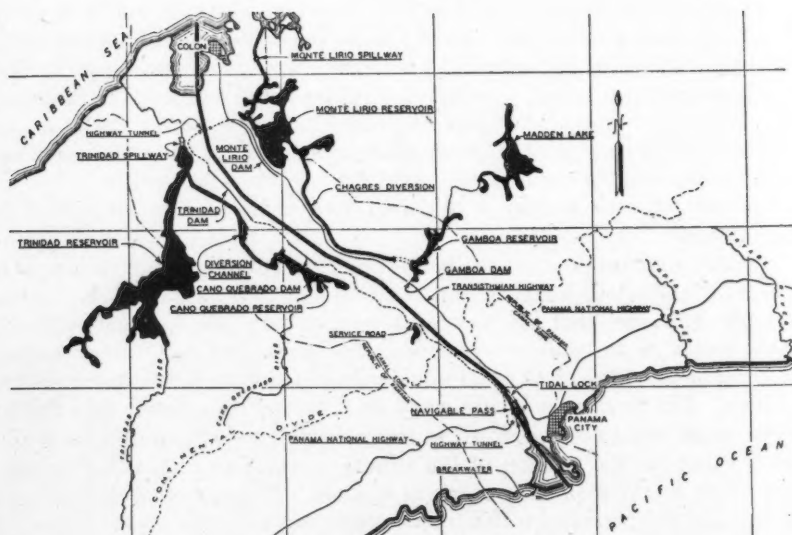
FIG. 7.—PLAN OF PANAMA SEA-LEVEL CANAL

already in place.   These installations would have to be duplicated at any
other sea-level route.   The plan of development for a sea-level canal at the
several routes was similar to that for a sea-level canal at Panama as described
in the next section.

PLAN OF DEVELOPMENT OF A SEA-LEVEL CANAL AT PANAMA

There are numerous possibilities for a sea-level canal in the Canal Zone and in the immediate vicinity;[7] the more favorable of these are shown in Fig. 8. The route that would cost the least and have other outstanding advantages is designated the Panama sea-level conversion route, since it follows generally the alinement of the present lock canal. The distinctive feature of a canal on the conversion route is that its construction would involve lowering the present canal to sea level, whereas this would not be the case if either the Panama parallel or one of the Chorrera canals were constructed.

ROUTES
1 CHORRERA-LAGARTO
2 CHORRERA-LIMON BAY
3 CHORRERA-GATUN
4 PANAMA PARALLEL
5 PANAMA SEA-LEVEL CONVERSION
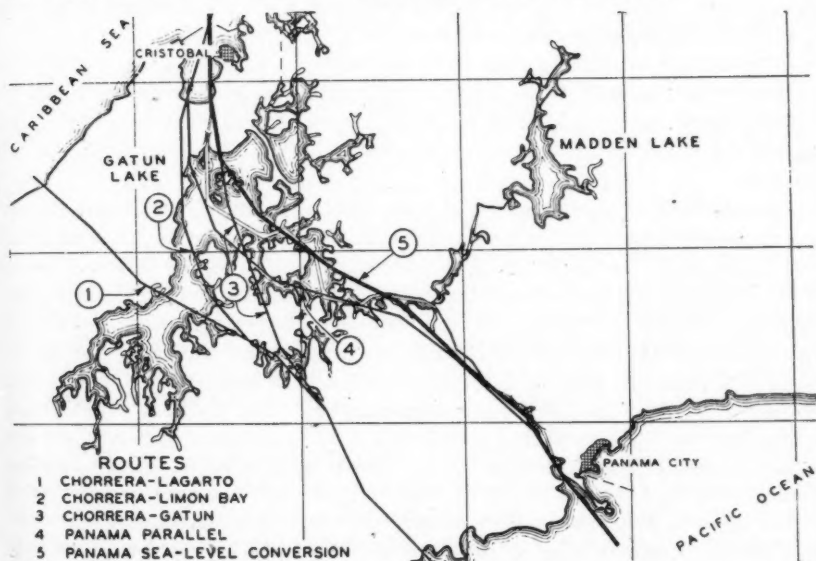
FIG. 8.—ROUTES IN CANAL ZONE AND VICINITY

In the Gatun Lake area, the Panama parallel canal would be separated from the existing lock canal by a barrier dam constructed from spoil material from the excavation for the new canal. From Gamboa south to Balboa Harbor it would follow an alinement separated from the present canal. Thus, the Panama parallel sea-level canal would be completely independent of the lock canal and, as would be the case if the Chorrera-Lagarto canal were constructed, both the existing canal and the new canal could be maintained and operated if this were thought necessary. There are no reasons arising either from capacity or security considerations that would justify the additional $800,000,000 for the Panama parallel sea-level canal. Any one of the Chorrera routes would be even more costly than the Panama parallel route and would be no more justified.

The alinement of the present canal and that of the proposed Panama sea-level conversion canal would be sufficiently separated at several beaches to make it possible to construct a substantial part of the latter in the dry, thus avoiding

[7] "Sea Level Plan for Panama Canal," by J. G. Claybourn, *Proceedings*, ASCE, February, 1947, p. 175.

interference with canal operations. The alinement improvements introduced in the conversion route would shorten the canal by 5.2 miles.

The Sea-Level Canal Channel

The channel of the Gaillard Cut in the present canal is deficient in width for two-way traffic involving large and unwieldy ships, and single directional transiting arrangements for such ships are in effect to avoid their encountering other ships in the cut. The delays and the inconveniences resulting from the special handling of this type of traffic have not thus far been seriously objectionable, but they would be with the further growth of traffic and this fact was borne in mind in designing the sea-level canal.

Currents up to 4.5 knots would be induced in a sea-level canal at Panama without tidal control by the Pacific tides which have a range up to 20 ft. Atlantic tides have a maximum range of 2 ft and would cause currents of 0.5 knot if the Pacific tidal effects were eliminated by control works at the Pacific entrance.

In establishing the design of the sea-level channel, a world-wide survey was made of comparable channels and canals. Several waterways having characteristics similar to the Panama sea-level canal, including the sea-level canals at Suez (Egypt) and Cape Cod (Massachusetts), were visited by members of the staff engaged on the studies. In addition, model investigations were made for The Panama Canal by the United States Navy at the David Taylor Model Basin at Carderock, Md., to determine, for both the lock and sea-level canals, the width, depth, and alinement requirements. For the sea-level channel determinations, various widths and depths of channel using both straight and bend channel sections were tested with currents up to 5 knots. Self-propelled, remote-controlled models of typical ships that transit the canal, operating at a wide range of ship speeds, were employed in testing the channels.

Two-way traffic of most of the ships that would be expected to transit the canal could be accommodated in a channel designed for the safe meeting of a standard "Liberty" ship by the largest naval craft or the largest commercial craft now afloat. The meeting and passing of the largest present-day naval craft and commercial vessels would be unusual and could be avoided by special transiting arrangements similar to those now in effect. Similar arrangements could be made for the transit of the largest ships expected to be built during the twentieth century.

In spite of considerations leading to the recommendation of the Governor of The Panama Canal (J. C. Mehaffey) that tidal currents in the canal be regulated, the channel was designed for safe navigation in currents up to 4.5 knots. This is made necessary by reason of the fact that the structures for the regulation of currents in the canal could be irreparably damaged by bombing and would have to be cleared from the channel. As the result of the studies, it was concluded that all shipping could safely transit the proposed Panama sea-level canal at any condition of current that would obtain up to the maximum of 4.5 knots; only an occasional unwieldy or low powered ship would be held for mean tide to avoid encountering high currents. Tug assistance could be

provided such ships to avoid delays and to provide increased safety of transit when thought necessary.

The channel standards adopted as the result of the model studies and other investigations are given in Table 3, as are the controlling standards of the present canal.

TABLE 3.—COMPARISON OF CHANNEL DIMENSIONS

Description	Depth (ft)	Width (at a depth of 40 ft) (ft)	Cross-sectional area (mini-mum) (ft)	Mini-mum sight distance (miles)	Maxi-mum deflection angle (degrees)	ANGULARITY	
						Total (degrees)	Per mile of canal length
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Present Panama lock canal.....	42	300	13,860	0.6	67[a]	598	11.7
Modernized Panama lock canal.	55	500	28,400	0.6	67[a]	642	12.5
Panama sea-level canal.........	60	600	36,800	1.5	26	117	2.5

[a] This applies to Gatun Lake; in Gaillard Cut the maximum angle is 30°.

The survey of the world waterways disclosed that channel depths are generally considered inadequate and that better ship controllability would result with more water under the keel.   The Carderock tests fully confirmed the latter conclusion.   The lesser depth proposed for the projected lock canal results from the lesser ship speed (8 knots) that would be prescribed in the restricted channel of a lock canal.   A ship speed of 10 knots was used to establish the depth of the projected sea-level canal.   The width of channel is referenced in each case to the 40-ft depth below the low water surface, which approximates the draft of the large vessels of the future.   This depth also establishes the point of revolution in setting bank slopes, thus minimizing variations in the surface width of the channel arising from differences in the slopes, which are fixed by the character of the bank materials.

The results of the model tests were interpreted with the assistance of the United States Navy and the pilots of the Panama Canal and of the Cape Cod Canal.   The advice and counsel of the Panama Canal pilots on all problems dealing with navigation were invaluable in the conduct of the studies.

SEA-LEVEL CANAL MODEL INVESTIGATIONS

George B. Pillsbury,[3] M. ASCE, has developed a procedure for computing currents in sea-level canals which, when applied to the proposed Panama sea-level canal, yielded results that were closely confirmed by Boris A. Bakhmeteff, Hon. M. ASCE, by independent computations and by model investigations. A model of the proposed sea-level canal at an undistorted scale of 1 : 100 provided methods of accurately determining conditions of flow in the uncontrolled waterway at all ranges of tide, of establishing the design, and of testing the tidal-regulating structures.   The maximum velocities in the unregulated

[3] "Tidal Hydraulics," by George B. Pillsbury, *Professional Paper No. 34*, Corps of Engineers, U. S. Government Printing Office, Washington, D. C., 1940.

canal 60 ft deep and 600 ft wide at 40-ft depth at various Pacific tides as computed and as observed in the sea-level canal model are given in Table 4.

TABLE 4.—VELOCITIES IN AN UNREGULATED SEA-LEVEL CANAL			
Tidal range (ft)	Percentage of time tides are exceeded	MAXIMUM VELOCITIES, UNCONTROLLED CHANNEL	
		Observed (knots)	Computed (knots)
(1)	(2)	(3)	(4)
6.....	99	2.1	2.1
10.....	80	2.7	3.0
13.....	50	3.3	3.5
16.....	20	3.8	4.0
20.....	2	4.4	4.5

TABLE 5.—SCHEDULE OF NAVIGABLE PASS OPENING IN A REGULATED SEA-LEVEL CANAL					
Permissible current velocity (knots)	HOURS PER DAY THAT THE NAVIGABLE PASS WOULD BE OPEN FOR TIDAL RANGES OF:				
	6 ft	10 ft	13 ft	16 ft	20 ft
(1)	(2)	(3)	(4)	(5)	(6)
1........	8.5	4.5	3.5	2.9	2.3
2........	20.2	9.7	7.2	6.0	4.9
3........	24.0	24.0	14.9	11.6	9.1
4........	24.0	24.0	24.0	24.0	19.8
4.5.......	24.0	24.0	24.0	24.0	24.0

TIDAL REGULATION FOR THE SEA-LEVEL CANAL

The majority (eight out of thirteen) of the Consulting Board appointed by President Theodore Roosevelt in 1905 to consider various plans for a canal at Panama prepared by the Isthmian Canal Commission in voting for a sea-level canal stated with respect to tidal control:

> "The plan [sea-level] proposed by the Board for the Isthmian transit will have twin tidal locks near the Pacific terminus, which if disabled, one or both [by enemy attack or sabotage] would still be usable (after removal of wreckage) for a part of each day (the period of spring tides) in each lunar month, and probably throughout the whole twenty-four hours the remainder of the lunar month (neap tides)."

If tidal regulation were to be omitted, occasional ships would need to be held at the canal entrances for a favorable tide, or tug assistance would need to be provided for their transit. The Carderock tests and experience in other waterways are conclusive in demonstrating that relatively few modern ships would require special transiting arrangements; the majority, having reliable power and rudder control, would be capable of transiting the Panama sea-level canal safely in currents up to 4.5 knots. Nevertheless, because of the serious consequence of collisions with the canal banks (which would be largely of rock), it was decided to provide tidal regulation for added safety to shipping. The loss of tidal regulation as the result of bombing of the regulating structures is a definite possibility, and is accepted as a condition of operation during wartime.

If the control of the Pacific tides were to be provided by tidal locks and a barrier dam and these were to be damaged by the enemy, then the canal would either have to be closed for repairs or the structures cleared from the channel to permit use of the canal as an open waterway. In the latter case, there would be an abrupt transition from slack-water navigation to navigation in currents

ranging up to 4.5 knots.   Prolonged closure of the canal for repairs would be intolerable, which suggests that the tidal-regulating structures should be planned to permit the rapid clearing of a "channel-way" in case they are damaged. This could be done by excavating an auxiliary channel which would normally be closed by an earth barrier dam that could be rapidly blasted to clear the channel when the locks were rendered unusable by bombing.   Obviously, an abrupt change from navigating a canal with currents completely regulated to one with currents of the order listed in Table 4 would be undesirable and this led to an investigation of other methods for providing tidal regulation.
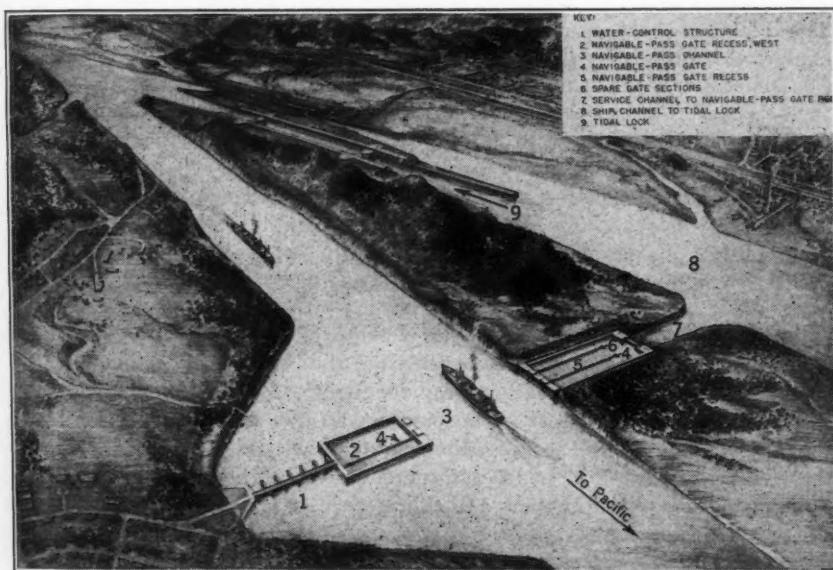
FIG. 9.—TIDAL-REGULATING STRUCTURES SHOWING THE NAVIGABLE PASS OPEN

A sudden transition from slack-water navigation to navigation in a completely unregulated waterway in the event of damage to the tidal lock could be avoided by supplementing the tidal lock with a navigable pass through which ships, as a matter of routine operation, could pass at any selected stage of the tide.   The pass would have retractable gates (Fig. 9) which could be opened rapidly to permit navigation without the need for lockage under any set of conditions of current desired, from 0.5 knot (Atlantic tidal currents) to the maximum. The gates of the pass would be of steel construction, and in the event of their damage could be readily removed from the channel.   Table 5 shows the hours daily that the pass would be open if the tidal currents were controlled to 1, 2, 3, 4, and 4.5 knots.   These data were determined by computations and confirmed by sea-level model tests.

It is probable that, when the sea-level canal is first placed in operation, the navigable pass would be opened only at mean tidal stages to limit channel currents to low values.   As operating experience is gained, the periods of opening

of the pass gates would be extended to permit higher currents in the canal. Tentatively, currents in the canal would be limited to 2 knots; this value was selected after a survey of operating experience in other waterways and as the result of the United States Navy tests at Carderock.   On this schedule for the opening of the pass gates, the canal would operate 32% of the total time as an open waterway.

A gated water-control structure would complement the navigable pass to assist in adjusting the water surface in the canal to extend the period of opening of the pass at each mean stage cycle of Pacific tide (Fig. 9.)   The gates of the water-control structure would be open only when the pass is closed to traffic. The tidal lock could accept ships at all stages of the tide and would be the sole avenue of transit when the pass is closed.

The sea-level canal model was an invaluable aid in planning the location, the arrangement, and the hydraulic features of the separate elements of the tidal-regulating works and in developing the operating schedule of the navigable pass.

One important phase of the model study was that of relating the friction factor of the model to the friction factor of the prototype channel.   This was accomplished by actual flow tests in Gaillard Cut with the culverts in the Pedro Miguel Locks wide open and discharging 22,000 cu ft per sec.

Slides in the Panama Sea–Level Canal[9]

A study of the major slides experienced in the period of the canal construction was of great assistance in comprehending the character and probable behavior of the materials to be encountered in any new excavation.   These slides, as was stated by the late George W. Goethals, M. ASCE, in 1916, were the result of an attempt to fix uniform slopes throughout the length of the canal, regardless of height of cut and character of materials.   General Goethals stated after the completion of the canal:

"* * * With the geological formation changing so frequently and so suddenly both in the direction of the Cut and up and down there is no possibility of any uniformity in slopes.   No uniformity of slopes could be maintained * * *."[10]

For a time after the first major construction slide took place, it was thought that the slopes would eventually stabilize themselves and that the volume of material ultimately to be removed would be less if the slides were allowed to run their course.   This proved not to be the result; moreover, the pressing need for opening the canal led to flattening certain of the slopes to prevent further blockage of the channel.   Fig. 10 represents for a specific location the design slopes now proposed for the Cucaracha formation, and shows the slope selected in the initial construction and the actual slope attained in this formation after failure.

Consideration was given in the design of slopes to dynamic loading that could be induced by bombing.   A study of the resistances of the materials to

    [9] "Panama Canal Slides," Rept. of the Committee of the National Academy of Sciences, *Memoirs*, National Academy of Sciences, Vol. XVIII, 1924.
    [10] "The Dry Excavation of the Panama Canal," by George W. Goethals, *Transactions*, International Eng. Cong., San Francisco, Calif., 1915.

dynamic (transient) loadings was made at Harvard University in Cambridge, Mass., under an investigational contract. The studies of the resistances of the materials to transient loadings and the tests of their residual strengths after failure will be of particular interest to engineers. The transient-load
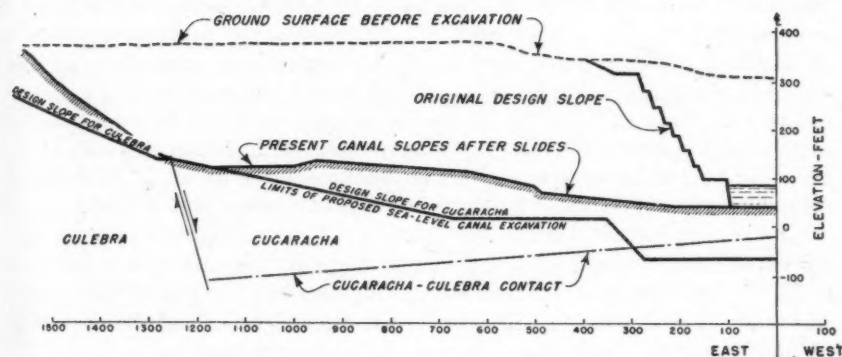
FIG. 10.—ORIGINAL AND PRESENT SLOPE DESIGN CRITERION (OLD CANAL STATION 1785+00)

testing apparatus developed at Harvard University offers promise as a valuable tool in developing fuller understanding of the behavior of soils and rocks under stress.

No allowance was made in the designs for slide failures resulting from dynamic loadings since it is unlikely that slides induced by the atomic bomb would fully block the canal. However, debris from cratering resulting from atomic bombing could block the channel, but widening the channel to overcome this possibility would require such a large amount of excavation that the risk of closure was accepted, since in any case the blocking materials, even though radioactive, could be removed in a few weeks at the outside. None of the conventional weapons could induce slides or throw up craters that would block the channel of either a sea-level or an improved lock canal.

CONVERSION TO SEA LEVEL

Studies of the conversion of the Panama lock canal to a canal at sea level prior to that under Public Law No. 280 contemplated the lowering of Gatun Lake in stages. One study planned for the lowering of Gatun Lake in seven stages. Philippe Bunau-Varilla,[2] the French engineer who negotiated the sale of the canal holdings of the French to the United States, urged the consulting board appointed by President Theodore Roosevelt to provide deep upper sills at each of the locks to facilitate the later conversion of the lock canal by stages to a sea-level canal. His plan was rejected on the grounds that major modification of the proposed twin locks would be inescapable in any case and that a third set of locks would be needed in order that two lanes of locks would be available for traffic at all times during the conversion period.

There is no question that the conversion of the existing locks would involve considerable risk both to shipping and to the integrity of the canal with only one lock lane available and that no plan should be accepted that would have less

than two lanes of locks available to shipping throughout the period of conversion. If the lowering of the summit lakes to sea level by stages is undertaken, new special twin conversion locks of minimum construction with lift to El. 53 are preferable to previously considered arrangements involving the progressive alteration of the existing locks for each stage of lowering using a new third set of locks with full lift to El. 85 to insure two lanes being available at all times. The proposed special twin conversion locks would be placed in operation and the existing locks abandoned when the first stage of lowering of Gatun Lake from El. 85 to El. 53 is accomplished—or after the completion of excavation to prepare the channel to carry traffic with the summit lake at El. 53. The next stage of lowering of the canal would be to El. 22, and finally the lowering would reach sea level; in each case excavation to provide necessary depth of channel would precede lake lowering. The twin conversion locks have low upper sills to accommodate traffic at all stages of canal elevation from El. 53 to El. 0. This plan and others considered for the stage lowering of the summit lakes were finally abandoned because of the attractiveness and the cheapness of the plan for the single-stage lowering of the summit lakes.

The conversion of the lock canal to sea level by lowering the summit lakes in a single operation would require dredges capable of excavating to a depth of 145 ft. For this purpose a cutterhead-type suction dredge would be used to excavate the softer materials and a chain-bucket type dredge, to excavate blasted hard materials. The suction dredge would have a booster pump mounted in a well on its ladder to effect the lift of the materials. The chain-bucket dredge would be similar to the conventional type used in mining of gold and tin except that the 2-cu-yd buckets proposed are considerably larger than those ordinarily used to depths of the order proposed. Dredges of the chain-bucket type worked successfully to depths of 128 ft in the mining of gold; however, the buckets generally have not exceeded $\frac{1}{2}$ cu yd. The Board of Consulting Engineers and others advising Governor Mehaffey on the current studies were unanimous in their opinion that deep dredging to effect single-stage conversion would be practical and economical.

In the deep-dredging method of conversion (single-stage lowering of the lakes), there would be a net saving of $130,000,000, principally through the omission of conversion locks. The actual lowering of the water surface to effect the conversion to sea level would be accomplished by progressive demolition of natural rock plugs and the removal of a temporary steel dam left to retain the Gatun and Miraflores lakes. The conversion could be effected with a traffic interruption of only 7 days.

Alinement improvements would permit a large part of the excavation to be done in the dry (750,000,000 cu yd) at a large saving in cost. There would be 318,000,000 cu yd of wet excavation classified as follows:

Cubic yards	Classification
9,400,000	Hard rock
97,700,000	Medium rock
46,600,000	Soft rock
165,000,000	Sands, gravels, and clays

Of this amount, approximately 132,000,000 cu yd would lie between depths of 85 ft and 145 ft below maximum water surface and would require the employment of special dredges.

FLOOD CONTROL IN THE SEA-LEVEL CANAL

Unless tributary inflows into the sea-level canal are controlled, there would be interruptions in service and occasional hazard to navigation. The most essential control is that of the Chagres River, the major tributary; this would
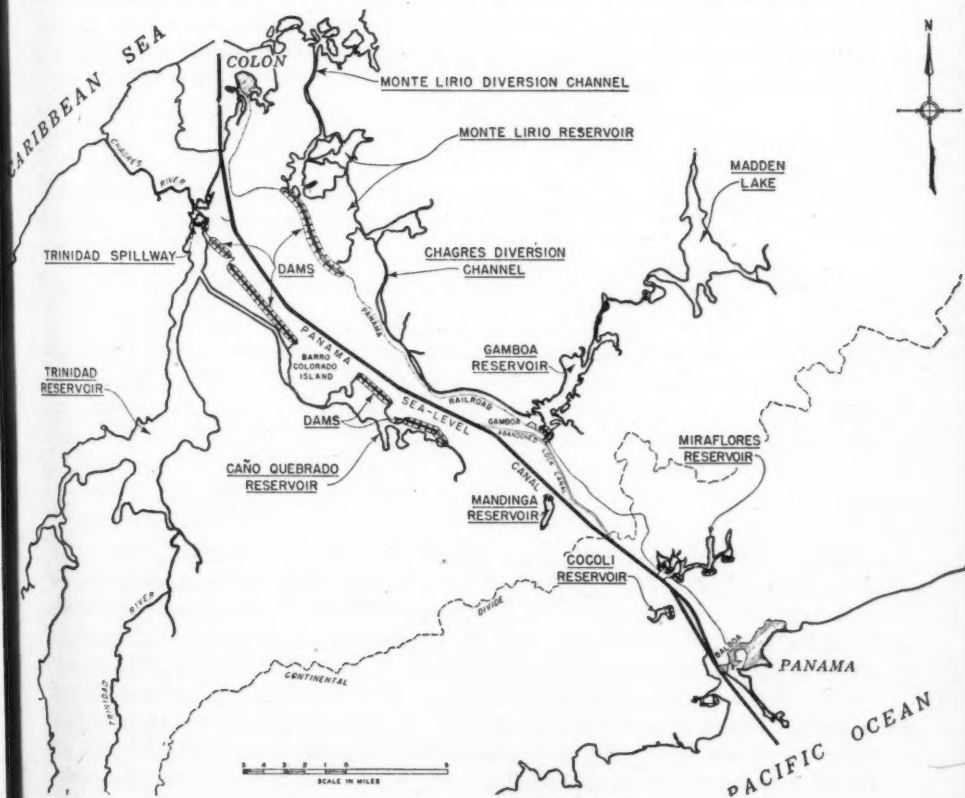
FIG. 11.—SEA-LEVEL CANAL FLOOD CONTROL SYSTEM

be accomplished by a dam constructed at Gamboa to operate in tandem with the upstream Madden Dam. The regulated outflows of the Chagres River would be excluded from the canal by diverting them to the sea through a short tunnel and a channel as shown in Fig. 11. The flows from all other tributaries on the east side of the canal north of the Chagres River would be intercepted and conducted to the Caribbean (Las Minas Bay) by a system of dams and the channels which convey the regulated flows from the Chagres. This control system, termed the "East Diversion," would have sufficient capacity to

divert all floods up to 25% larger than the maximum flood of record. Emergency outlets into the canal are provided for larger floods.

"The West Diversion" system of dams, reservoirs, and channels would exclude the entry of flows into the canal from all the important tributaries west of the canal and north of the Continental Divide by diverting them into the channel of the lower Chagres through an outlet and spillway west of the present Gatun Dam.

South of the Continental Divide there are a number of small tributaries (maximum drainage area 13 sq miles) which could contribute undesirable flood inflows if not regulated. Diversion being impracticable because of the topography and the developments in the area, a system of regulating reservoirs as shown in Fig. 11 would be provided, the corresponding drainage areas being:

Reservoirs (see Fig. 11)	Square miles
East Diversion—	
Madden Lake	393
Gamboa Reservoir	127
Monte Lirio Reservoir	180
West Diversion—	
Caño Quebrado Reservoir	166
Trinidad Reservoir	322
Small Reservoirs—	
Mandinga	10.5
Cocoli	11.5
Miraflores	30
Uncontrolled	118
Total	1,358.0

The proposed combination of reservoirs and diversion systems would control 1,240 sq miles of the total of 1,358 sq miles of area tributary to the canal between Gatun and Balboa. Of the uncontrolled drainage areas, the largest would not exceed 6 sq miles. The flood contribution from the uncontrolled areas would neither inconvenience navigation nor be a hazard to ships in transit.

The flood-control dams would be constructed from excavation spoil except where the length of haul would make local borrow cheaper. The east and west diversion dams would be low structures of earth and rock spoil mounted on broad platforms of fill constructed of excavation spoil placed in the wet, thus insuring very conservative loadings where the foundations are muck and soft clays. The earth and rock flood-control structures would be readily repairable in case of damage by enemy bombing.

Sea-Level Canal Construction Plan

It is planned to complete the construction of the sea-level canal in 10 years. The construction plan adopted in the report to Congress provides for the use of large shovels and draglines (25 yd or larger) dumping into scows for the haul of dry excavation, totaling 750,000,000 cu yd, to disposal areas in Gatun Lake. Wet excavation to customary depths would be performed by conventional

dredges, but for depths beyond the range of this equipment, special dredges capable of excavating to 145 ft below the water surface would be required. Vehicular tunnels under the canal would be constructed at the Atlantic and Pacific ends to aid construction and for cross-channel access thereafter. Total cost of the sea-level canal is estimated at $2,483,000,000.

CAPACITY OF THE PANAMA SEA-LEVEL CANAL

The capacity requirements for the year 2000 on peak days would be 69 transits. For planning purposes, ship speeds in the sea-level canal were established at 12 knots ground speed traveling with the current and 8 knots against current. A conservative spacing of 1.5 nautical miles between ships was selected after a survey of the practices of other waterways and with the advice of Panama Canal pilots. The daily capacity of the sea-level canal, based on a 16-hour operating schedule for both the tidal lock and the navigable pass, and assuming the latter would be opened to limit currents up to 2 knots, would be 116 transits daily. The average transit time in the sea-level canal would be 4.5 hours. This compares with an average transit time of 8 hours in the present canal.

SECURITY OF THE PANAMA SEA-LEVEL CANAL

The tidal-regulating and flood-control structures of the sea-level canal would not be essential to its safe operation; hence their damage or destruction would result only in temporary interruptions to traffic. An adequate flood-warning system would insure against ships being caught in the channel at the time of incidence of large flood inflows in the event of damage to the flood-control structures. The flood-impounding structures being of earth and rock construction would be highly resistant to bombing but, because they are not absolutely vital to the operation of the sea-level canal, the costly treatment necessary to make them resistant to the atomic bomb would not be warranted. If damaged by bombing, the flood-control structures could be readily repaired and restored to service.

The tidal-regulating works could not be made resistant to modern weapons. Their loss by enemy action would be the loss of convenience to shipping and a lessening of safety in transit, particularly for ships with low power and poor rudder control. The risks in transit of such ships could be avoided by having them await a favorable tide or by providing them with tug assistance. The tidal lock, if heavily damaged, would be a mass of debris that would be difficult to remove. If the tidal lock were contaminated by radioactive particles as the result of atomic bombing, it would probably have to be abandoned. In either case the navigable pass could be used for transiting traffic. If the pass gates were damaged, they could be removed from the channel in a few days to a few weeks, depending on the extent of damage to the channel-way. Thereafter, the canal would operate as an open waterway.

None of the conventional weapons known today could induce slides or blockage of the channel by cratering that would close the sea-level canal to traffic. However, the canal could be blocked for limited periods as the result of an atomic weapon attack. It is estimated that a crater blockage

of the canal would require a few weeks, at the most, for the clearing of a traffic lane, even though radioactive contamination would delay the initiation of the removal work. The shielding of the dredges and other excavation equipment to protect crews would be required, and this is considered practicable.

An attack employing conventional or atomic weapons could do great damage to the housing and administrative facilities of the canal and could result in great loss of life but, disastrous as the loss of life would be, the operation of the sea-level canal could go on uninterrupted since ships in an emergency could transit the canal through the navigable pass under the pilotage of their masters.

ACKNOWLEDGMENTS

The papers of this Symposium summarize the major studies, except those pertaining to security, made by the staff of The Panama Canal and its consultants and collaborators. Those who contributed to the studies are too numerous to name, but acknowledgment of their contribution is made in the report of the Governor of The Panama Canal under Public Law No. 280. The report consists of text and a folio of eight plates, supplemented by eight annexes and twenty-one appendixes. Appendix 21 of the report contains a complete bibliography for the Isthmian Canal studies. Particularly valuable assistance was provided by the Bureau of Yards and Docks and the Bureau of Ships, Department of the Navy; the Chief of Engineers, Department of the Army; and the Atomic Energy Commission. All major features of the study were reviewed by a Board of Consulting Engineers composed of Boris A. Bakhmeteff and William H. McAlpine, Hon. Members, ASCE; Hans Kramer, John J. Manning, Hibbert M. Hill, and Joel D. Justin, Members, ASCE; and until his retirement in September, 1946, Ben Moreell, Hon. M. ASCE. The writer was in immediate charge of the studies under J. C. Mehaffey, the Governor, The Panama Canal, who was charged by Public Law No. 280 with the responsibility for the studies.

TRAFFIC AND CAPACITY

By Ralph P. Johnson[11] and Sydney O. Steinborn,[12]
Assoc. Members, ASCE

Synopsis

A forecast of future vessel movements through the Panama Canal and a study of the important characteristics of this traffic indicate that 16,660 vessels of all types may be expected to transit a Panama canal in the year 2000. The average daily traffic would be 46 vessels; the peak-day traffic for which the canal should be designed is 69 vessels. By about 1960, the capacity of the present canal, during periods of lock overhaul, would be insufficient to accommodate traffic without undesirable delays on peak days. With modifications principally to eliminate the periodic need for closure of one of the twin locks for overhaul, the capacity of the present canal could be increased to meet all commercial requirements until the end of the twentieth century. Because of the limited size of the lock structures, certain large naval and commercial vessels would be unable to transit the canal.

The construction of new and larger locks or the conversion of the present canal to sea level would create sufficient capacity to handle all traffic until well beyond the year 2000.

Introduction

The studies under Public Law No. 280, Seventy-ninth Congress, included investigations of the methods of increasing the capacity of the Panama Canal to meet the future needs of interoceanic commerce and national defense. This aspect of the studies required a prediction of the volume and characteristics of canal traffic at some future date which, for planning purposes, was selected as the year 2000. As stated in the first Symposium paper, the prediction was based on a recent evaluation of future Panama canal commercial traffic made by Professor Kramer of the University of Pennsylvania, who was engaged as a traffic consultant by The Panama Canal. The capacities of the present Panama Canal, of two improved Panama lock canals, and of a Panama sea-level canal were then analyzed and their capacities were compared to the estimated design peak-day demand for transits to determine when demand would exceed capacity. These comparisons provided an index of the date at which new facilities would be needed at Panama and of the relative adequacy of the proposed canal improvements.

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Ocean-Going Commercial Traffic

The largest group of vessels transiting the Panama Canal is that which pays tolls. It consists of merchant vessels, foreign naval vessels, privately-owned dredges, yachts, and similar craft. Tolls-paying vessels of more than 300 tons are considered to be ocean-going commercial vessels. In peacetime these make up about 78.5% of all transits, and carry nearly all the commercial cargo passing through the Panama Canal. Tolls-paying vessels of less than 300 tons are engaged principally in local trade and carry an insignificant volume of the commercial cargo.

Future Tonnage of Ocean-Going Commercial Traffic.—Professor Kramer's estimate of future Panama Canal traffic is concerned only with cargo carried in ocean-going commercial vessels. His forecast is based on the following assumptions:

a. No widespread wars or serious political disturbances will take place during the period covered by the forecast.

b. The effects of unforeseeable changes in the sciences, such as development of atomic power and of air transport, are not appraised.

c. Foreign trade will develop as the United Nations organization achieves stature and as trade restrictions are reduced.

d. The United States will maintain a substantial merchant marine in accordance with the mandates of the merchant marine acts of 1920, 1928, and 1936, and of the Ship Sales Act of 1946.

Professor Kramer's forecast, termed an "economic-statistical projection," correlates Panama Canal commercial traffic with the trend in United States national income as based on future United States population and future per capita income. In this forecast, an estimate of future United States ocean-borne imports, exports, and intercoastal trade was made according to the past relationships between national income and these three factors. Next, the past relationships of Panama Canal commercial traffic to these factors were determined. The relationships established from these estimates were used to derive estimates of future intercoastal, United States import, and United States export traffic through the canal. A fourth factor of Panama Canal commercial traffic, "foreign-to-foreign" shipments, was determined on a percentage basis from past records. The summation of the four components of Panama Canal commercial traffic, intercoastal, United States import, United States export, and "foreign-to-foreign" shipments, resulted in the following net-vessel tonnage forecast of predicted growth of ocean-going traffic (see Fig. 1):

Year	Net vessel tons
1950	36,185,000
1960	44,929,000
1970	54,639,000
1980	65,207,000
1990	75,502,000
2000	86,312,000

This prediction is considered to represent the probable maximum load that will be put on the canal in the future.

For purposes of comparison, earlier predictions of Panama Canal traffic, made by Emory R. Johnson in 1912 and Grover G. Huebner in 1936—both of the University of Pennsylvania, are shown in Fig. 12 As a matter of interest, the traffic history of the Suez canal is also shown.

Conversion of Ocean-Going Commercial Tonnage to Transits.—The number of ocean-going commercial transits expected annually is obtained by dividing Professor Kramer's estimate of yearly tonnage by the estimated average net vessel tonnage per ocean-going commercial transit. The estimate of the average tonnage of ocean-going commercial vessels is based on past experience at Panama and on long-term trends developed at the Suez Canal.

The average net vessel tonnage of oil tankers transiting the Panama Canal has been about 30% higher than that of vessels carrying other types of cargo (general cargo vessels). Also, the relative volume of tanker traffic in the past is very large when compared to its future relative volume as predicted by Professor Kramer. For these reasons general cargo traffic at Panama was analyzed separately from tanker traffic.
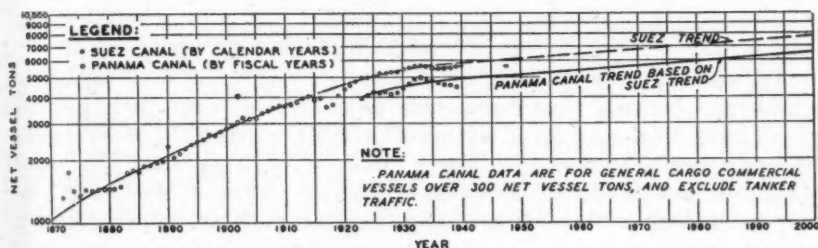
Fig. 12.—Average Net Vessel Tonnages of General Cargo Ships

The average tonnage of ocean-going general cargo vessels at Panama was determined for 17 "normal" years (from 1923 to 1939) during which traffic was relatively independent of influences arising from World War I and World War II. Because the Panama Canal statistics covered so few years, no well-defined trend could be detected in a plot of these data. Accordingly, the plot was compared with the average tonnage of vessels transiting the Suez Canal in the years from 1870 to 1939 (Fig. 12). The trend of Suez data is well defined and may be extrapolated to the year 2000. The average tonnage of vessels transiting at Suez was thus assumed to increase from 5,600 tons in 1939 to 7,800 tons in the year 2000. As the Panama Canal data are roughly parallel to those at Suez, the Panama Canal trend was assumed parallel to that of Suez and the average tonnage of general cargo vessels transiting at Panama was assumed to increase from about 4,500 tons in 1939 to 6,600 tons in the year 2000. Because of the predicted relative insignificance of tanker traffic, the trend in the average tonnage of general cargo vessels is assumed to be applicable to all ocean-going commercial traffic. The number of ocean-going commercial transits for any year up to the year 2000 is then obtained by dividing

the estimated tonnage for that year (Fig. 1) by the estimated net vessel tonnage per transit for the corresponding year (Fig. 12). The predicted increase in the number of ocean-going commercial transits is shown in Fig. 3 and in Col. 2, Table 6.

TABLE 6.—PREDICTED GROWTH OF VESSEL TRANSITS THROUGH THE PANAMA CANAL

Year	Ocean-going commercial vessels	Small commercial vessels	Tolls-free vessels	Total transits	Average daily transits
(1)	(2)	(3)	(4)	(5)	(6)
1950	6,959	567	1,339	8,865	25
1960	8,199	669	1,577	10,445	29
1970	9,486	773	1,825	12,084	34
1980	10,796	880	2,077	13,753	38
1990	11,947	974	2,298	15,219	42
2000	13,078	1,066	2,516	16,660	46

In the absence of any reason for change, it is expected that in peacetime ocean-going commercial vessels will comprise the same percentage of canal traffic in the future that they have in the past—namely, 78.5%.

TRAFFIC OF SMALL COMMERCIAL VESSELS

Tolls-paying vessels of less than 300 tons—small commercial vessels—are considered separately from ocean-going commercial vessels. Although these vessels make up about 6.4% of all transits, they carry only 0.1% of the commercial cargo passing through the canal. However, they are a tax on the capacity of the canal in that they are vessels that must be handled and therefore they cannot be disregarded. It is expected that the traffic of these vessels will increase at a rate corresponding to that of ocean-going commercial traffic and thus will continue to comprise 6.4% of total Panama Canal transits in peacetime. The predicted growth in the traffic of small commercial vessels is shown in Col. 3, Table 6.

TOLLS-FREE TRAFFIC

International treaties and federal legislation permit several classes of vessels to transit the canal without charge. They consist of United States Navy combat and auxiliary vessels, United States Army vessels, cargo-carrying or other vessels operated in the service of the United States Government, warships of the Republic of Colombia, Panamanian Government vessels, vessels transiting solely for the purpose of repair at Canal zone shops, and Panama Canal operational and maintenance equipment. In peacetime, these tolls-free vessels account for 15.1% of all transits. There is no indication that the relative volume of this traffic is changing and therefore the 15.1% factor is used to determine the future peacetime volume of all tolls-free traffic (Col. 4, Table 6).

FUTURE TRAFFIC TOTALS

Total Panama Canal traffic in the future will consist of all commercial traffic and all tolls-free traffic whose predicted values are shown in Fig. 2 and in

Cols. 5 and 6, Table 6. These totals are based on percentage factors derived for peacetime years. The volume of military combat and cargo traffic in future wars is indeterminate, but experience during World War II indicates that commercial traffic volume would drop and tend to offset the increase in military traffic. Because there is no assurance that the drop in commercial traffic will completely offset the rise in military traffic, a canal should have some excess capacity to meet the unknown demands that may be put on it during wars.

TRAFFIC CHARACTERISTICS AFFECTING CAPACITY

Variations in Daily Traffic.—The traffic characteristic which controls the capacity requirement of the Panama Canal is the daily variation in the demand for transits. Annual and seasonal variations in traffic volume are so much less than the daily variation that they are of no significance in establishing the capacity requirement of the Panama Canal. Daily variations in traffic volume have been extreme; immediately before World War II, however, the number of transits on the peak day of the year had decreased and appears to be leveling off at 160% of the number of transits on the average day.

The frequencies with which peak traffic days are expected to occur are shown in Table 7. It is evident that very few delays to shipping would occur

TABLE 7.—PREDICTED FREQUENCY OF PEAK DAYS, PANAMA CANAL

PEAK-DAY CONDITIONS		Days per year in which peak-day traffic would be exceeded
Percentage of average daily traffic	Transits in the year 2000	
130.............	60	45
140.............	64	24
150.............	69	9
160.............	73	0

if the capacity requirement were established so as to provide for traffic peaks 50% in excess of the daily average. With this daily capacity, traffic would be held over to the next day on an average of only 9 days a year, and, in the year 2000, delays would be of such an order that about 20 vessels would be held up a total of about 400 ship-hours with 24-hour operation of the canal in effect. This service is compatible with the high standards expected of the Panama Canal. Therefore, days in which the traffic is 50% in excess of the average are used to establish the capacity requirement of the Panama Canal and are called "design peak days." In the year 2000, this requirement is the capacity to transit 69 vessels of all types in 24 hours; this requirement and the requirements for other years are shown in Fig. 1.

Direction of Traffic.—In the past, 52% of the yearly ocean-going commercial transits passed through the canal from the Atlantic to the Pacific and 48% transited from the Pacific to the Atlantic. From this, it is assumed that, in effect, future traffic will be evenly divided with respect to direction. The assumption applies only to annual traffic; on individual days traffic may be considerably unbalanced and traffic in any one direction may be nearly three times

as great as traffic in the opposite direction.. Under certain conditions, the expected maximum unbalance of traffic on peak days would reduce the maximum capacity of a lock canal slightly, but would have no significant effect on the capacity of a sea-level canal. The effect of unbalanced traffic on the capacities of lock canals is described subsequently in this paper.

Vessel Length.—The number of vessels which can be placed in a given lock chamber at one time is controlled by their lengths. This has an important effect on the capacities of the lock structures in a lock canal or in any tidal locks used in a sea-level canal. Using the records of the Panama Canal. (from 1921 to 1940), traffic has been grouped into the vessel-length categories shown in Table 8.

The records show that the composition of traffic by vessel lengths has varied little from year to year. However, studies made of the relations of net vessel tonnage, beam, length, and tonnage per foot of length, indicated that the predicted increase in average net vessel tonnage would be accompanied by a slight increase in vessel length. Furthermore, it was indicated that a 10-ft increase in the average length of all vessels

TABLE 8.—COMPOSITION OF ALL TRAFFIC THROUGH THE PANAMA CANAL FROM 1921 TO 1940 BY LENGTH OF VESSEL

Length (ft)	Percentage	Transits
> 500	6.2	8,004
400 to 499	47.3	60,870
300 to 399	20.0	25,806
200 to 299	7.3	9,395
< 200	19.2	24,535
Total	100.0	128,610

would be sufficiently conservative for estimating the capacities of lock structures in the year 2000. It was assumed that this increase would be uniform from the present to the year 2000. Table 9 shows the percentages of vessels that may be expected in each length category for the years 1960 and 2000.

Experience at Panama indicates that the composition of traffic by length of vessel on peak days is very similar to the average annual composition. This fact is of particular significance in establishing the capacity requirements of lock structures which should be able to accommodate shipping on design peak days.

TABLE 9.—PREDICTED COMPOSITION OF ALL TRAFFIC THROUGH THE PANAMA CANAL BY LENGTH OF VESSEL

Vessel length (ft)	PERCENTAGE OF TRANSITS		Transits, year 2000
	1960	2000	
> 500	8	11	1,832
400 to 499	46	45	7,498
300 to 399	20	19	3,165
200 to 299	7	7	1,166
100 to 199	8	8	1,333
< 100	11	10	1,666
Total	100	100	16,660

DIMENSIONAL LIMITATIONS OF PANAMA LOCK AND SEA-LEVEL CANALS

The maximum size of vessel that could transit a lock canal or that could pass through the tidal lock of a sea-level canal would be limited by the dimensions of the lock chambers. At present only two commercial vessels, the *Queen Mary* and the *Queen Elizabeth*, are too large to be received in the existing 110-ft by 1,000-ft lock chambers. These vessels are used in the North

Atlantic service and would not be expected to operate through the canal. Until the approach of World War II, the lock chamber dimensions of the Panama Canal were accepted as a limitation in the design of Navy vessels. However, several warships constructed or modified during the war exceed the dimensions of the present locks and are therefore unable to utilize the canal. The proposed lock chamber dimensions of 200 ft by 1,500 ft for either a high-level lock of a lock canal or the tidal lock of a sea-level canal would permit the transit of the largest commercial vessels expected up to the year 2000 and would allow a considerable increase in the size of naval vessels without prohibiting their transit. The 750-ft-wide navigable pass of the sea-level canal would permit the transit of any type of vessel of any size likely to be built in the twentieth century.

Capacity of the Present Panama Canal

The capacity of a lock canal is established by the capacity of its locks as reduced by any conditions which affect the availability of the locks for the transit of vessels or which prevent vessels from entering the locks. Examples of such conditions are the occurrence of fog, restrictions fixed by channel dimensions, or lay-up of the locks for overhaul.

The length of the lock chamber, as related to lengths of future vessels, determines the average number of vessels per lockage. Using the length composition shown in Table 9, it was found that, under ideal conditions, an average of 1.8 vessels could be transited simultaneously in the existing locks (110 ft wide by 1,000 ft long). For conservatism, the estimated number of the vessels per lockage was reduced to 1.5.

The time interval between successive lockages in the same direction has been determined from time and motion studies of operations on the existing locks, and amounts to 45 min with double-culvert filling and emptying of the lock chambers. Thus, operating 24 hours daily under ideal conditions, the present locks could accommodate a total of 96 vessels.

An unbalanced flow of traffic in either direction would require some changes in the direction of lockages. The time interval between successive multiple-lift lockages in opposite directions in the same lane is greater than that for successive lockages in the same direction because, in the first instance, the incoming vessel must wait until the outgoing vessel has cleared the lock structure before entering. The longer interval results in a loss of capacity; in the present canal, however, the physical limitations of Gaillard Cut which necessitate one-way traffic at night would give ample opportunity to balance traffic and capacity losses from this cause would not occur.

The frequency with which fogs occur in the Panama Canal is such that the dependable daily capacity of either a lock or a sea-level canal must be based on the assumption that fog is a daily occurrence. Fogs in Gaillard Cut are most frequent during the rainy season and occur generally between midnight and 8 a.m., with an average duration of 4 hours. At present, it is hazardous for a ship to enter Gaillard Cut whenever there is a possibility of fog. This possibility exists during the wet season and, thus, for 8 months a year Gaillard Cut is closed 8 hours a day. The effect of this closure is a loss in capacity of 32 vessels daily.

Traffic through Gaillard Cut is limited to one direction in the case of un-wieldy vessels or of vessels with more than an 80-ft beam, for vessels carrying explosive cargoes, and for all vessels at night. This restriction results in a loss of capacity of 6 vessels daily.

The locks are overhauled biennially at Panama, alternating between the Pacific and Atlantic locks. Thus each lock structure is overhauled once every 4 years. The necessary repairs and replacements are completed in about 4 months, and during that time only one lock lane is available. For most of the overhaul period, only a single culvert is available for filling and emptying the lock chambers, and, therefore, the time interval between successive lockages in the same direction is lengthened to 56 min. The daily lock capacity during overhaul is then 39 vessels.

Since only one lock lane is available during overhaul, the direction of lock-ages must be changed to permit transits in both directions. Two such changes are assumed to occur daily and this results in a daily loss in capacity of 3 vessels during periods of overhaul.

Overhaul is performed in the dry season when fogs are rare and therefore there are no losses in capacity because of fogs during overhaul periods. Since traffic during the overhaul period is essentially single lane traffic, there are no losses in capacity at such times resulting from traffic restrictions in Gaillard Cut.

Dependable Daily Capacity of Present Canal.—The estimated capacity of the present canal, based on the factors previously described, is developed in

TABLE 10.—DAILY CAPACITY OF THE PRESENT
PANAMA CANAL (NUMBER OF VESSELS)

Line	Description	Outside of overhaul periods[a]		During overhaul periods[b]	
1	Theoretical capacity......................		96		39
	Capacity Losses Resulting from—				
2	Changes in direction of transit...........	0		3	
3	Possibility of fog in Gaillard Cut.........	32		0	
4	Restricted channel in Gaillard Cut........	6	−38	0	−3
5	Net capacity.............................		58		36

[a] Operation outside of overhaul periods; maximum theoretical capacity of two lock lanes, with double-culvert filling and 1.5 vessels per lockage. [b] Operation during overhaul periods; four consecutive months every two years; maximum theoretical capacity of one lock lane, with single-culvert filling and 1.5 vessels per lockage.

Table 10. Table 10 shows that the daily capacity of the present canal is a minimum during periods of lock overhaul. Each lock overhaul lasts 4 months, and it is neither reasonable nor practical to delay vessels over such an extended period. Therefore, the dependable daily capacity of the existing canal is taken at its minimum daily capacity during overhaul—or 36 vessels.

CAPACITY OF THE PRESENT CANAL WITH MINOR IMPROVEMENTS

Table 10 indicates that the greatest improvement for increasing the capacity of the present canal would be to eliminate the necessity of taking one lock lane out of service during overhaul periods. Such an improvement is entirely feasible and could be accomplished by modification of the lock gates, gate settings, and culverts. Lock filling and emptying would then be done with a

single culvert for each lock lane while the valves in the third culvert were being serviced.   This operation would take place in the fog-free dry season to permit 24-hour operation of the locks.   Under these conditions, the net capacity of the canal would be the capacity of both lock lanes, each in single-culvert operation, less losses resulting from channel restrictions imposed by Gaillard Cut—or 70 vessels daily.   This is in excess of the 58-vessel daily capacity of the present canal outside of overhaul periods, and the latter capacity would become the controlling capacity of the canal.

To increase the dependable daily capacity of the canal to more than 58 vessels, additional improvements are necessary.   The most economical of such improvements is the provision of tie-up stations in Gaillard Cut.   These would permit transits to continue until the actual occurrence of fog and it is estimated that they would increase the dependable daily capacity of the canal to 65 vessels.

The foregoing modifications constitute the minimum plan of improvement of the existing canal and would provide adequate capacity to handle traffic until close to the end of the twentieth century.

If a further increase in capacity is desired, it can be obtained by providing the navigation aids that would permit one-way navigation in fog.   With navigation in fog the daily capacity of the canal outside of overhaul periods would be 96 vessels less losses caused by channel restrictions imposed by Gaillard Cut—or 86 vessels.   However, the daily capacity of the canal during the modified overhaul period would be 70 vessels.   Since this is the smaller capacity, it is controlling, and represents the increase in capacity achieved by use of navigation aids.   This capacity slightly exceeds the expected design peakday demand for transits in the year 2000.

Capacity of a Completely Modernized Panama Lock Canal

Complete modernization of the present Panama lock canal would provide for the construction of two new locks (200 ft by 1,500 ft) at each end of the canal, abandonment of the existing locks, widening of Gaillard Cut permitting two-way navigation at night, raising of Miraflores Lake to the level of Gatun Lake to obtain a Pacific summit-level anchorage, and the construction of tie-up stations throughout Gaillard Cut.   Using the length composition of vessels for the year 2000 shown in Table 9, an average of 2.9 vessels, under ideal conditions, could be locked through the proposed locks in tandem.   For conservatism, however, it was assumed that the locks could accommodate an average of 2.4 vessels per lockage.   With the time interval between successive lockages in the same direction estimated to be 57 min, the theoretical maximum capacity of the locks would be 120 vessels daily.   Losses caused by an assumed single change in direction to provide for unbalanced traffic would reduce this to 118 vessels.

Without tie-up stations and summit-level anchorages, the loss in capacity in the 8-hour period during which Gaillard Cut would be closed because of the occurrence or possibility of fog would amount to 40 vessels daily.

The provision of a Pacific summit-level anchorage to match that at Gatun would permit vessels to lock-up to the summit level when Gaillard Cut was

closed because of fog. With this operating procedure, the loss in capacity due to fogs would be reduced from 40 vessels daily to 25 vessels daily.

With tie-up stations, vessels could proceed through Gaillard Cut until the actual occurrence of fog. The loss in capacity due to fogs would then be further reduced and would amount to 15 vessels daily. The dependable daily capacity of a completely modernized lock canal would therefore be 103 vessels (118 minus 15)—well in excess of the requirement for the year 2000.

The time required to transit, if two new and larger locks were constructed at each end of the canal, would be 7.25 hours, as compared with 8 hours in the present canal under normal operations and 8.5 hours during overhaul periods.

Capacity of the Panama Sea-Level Canal

The capacity of the Panama sea-level canal is established by the conditions under which its tidal-regulating structures are assumed to operate, by the assumed vessel speed and spacing, and by the duration of fogs along the channel.

The tidal-regulating structures of a sea-level canal would consist fundamentally of a tidal lock and a navigable pass. Shipping would utilize the tidal lock when the tidal head between the canal and the Pacific Ocean was large, and the navigable pass when tides were at or near their mean.

The amount of tidal regulation necessary to limit the channel currents to a selected maximum value depends upon the range of the Pacific tides. (This subject is treated subsequently in the fourth Symposium paper.) The capacity of the tidal-regulating structures increases with increased use of the navigable pass—that is, inversely with the tidal range. The dependable capacity of the sea-level canal using tidal regulation has been taken as its capacity on days when 20-ft tides would prevail. Since the navigable pass would be open for longer periods 98% of the time, the assumption is extremely conservative.

The tidal-regulating structures would be designed to control the currents to any value between the maximum velocity resulting from the Atlantic tides alone (about 0.5 knot) and the maximum velocity of about 4.5 knots produced in an uncontrolled waterway by the combination of both Pacific and Atlantic tides. A maximum allowable current of 2 knots has been selected in these studies as the basis upon which to establish the dependable capacity of the sea-level canal.

Vessel Speed and Spacing.—The capacity of a channel depends on vessel speeds and vessel spacings; in these studies an average water speed of 10 knots has been assumed. The factors that bear on the safe distance between vessels traveling in the same direction are their speed, size, power, loading, controllability, operating dependability, channel dimensions, alinement, visibility, weather, and currents. In consideration of these factors, the spacings shown in Table 11 for vessels more than 300 ft long have been selected for the purpose of estimating capacity. Vessels less than 300 ft long can be interspersed with larger vessels spaced in accordance with Table 11 without impairing navigational safety. If vessels were to travel at higher speeds, greater spacing would be required for safety and the net effect would be a reduction in capacity. (Spacings in Table 11 are for vessels longer than 300 ft, traveling at an average water speed of 10 knots.)

Capacity of Tidal Lock.—The daily capacity of the tidal lock depends on

the hours of operation, the time interval between lockages, and the number of vessels that would be assembled in the lock chamber at one time. The tidal locks would be the same size as the locks of a modernized lock canal and 2.4 vessels could be handled during each lockage. For 24-hour operation and an average time interval between lockages of 40 min, the daily capacity of the tidal lock would then be 86 vessels.

Capacity of Navigable Pass.—When open, the navigable pass would give full freedom to the flow of traffic and its capacity would be the same as that for the sea-level channel. The daily capacity of the pass thus depends on the length of time it would be open for navigation and on the adopted vessel spacing (Table 11). If the tidal-regulating structures were operated to limit channel currents to 2 knots, the pass would be open about 4.9 hours during each day having 20-ft tides. The daily capacity of the navigable pass would then be 88 vessels. An increase in the maximum allowable current would permit the pass to remain open for longer periods, and the capacity of the pass would thereby be increased. The daily capacity of the navigable pass for various tides and allowable current is shown in Table 12.

TABLE 11.—VESSEL SPACING

Channel current (knots)	Spacing (nautical miles)
0	1.0
2	1.5
4 to 5	2.0

TABLE 12.—DAILY CAPACITY OF NAVIGABLE PASS

Maximum allowable current (knots)	Daily Capacity in Vessels for a Tidal Range of:		
	20 ft	16 ft	13 ft
1	53	66	80
2	88	110	130
3	140	180	240
4	260	320	320

TABLE 13.—CAPACITY OF SEA-LEVEL CANAL WITH TIDAL REGULATION (16-HOUR OPERATING DAY)

Maximum allowable current (knots)	Number of Vessels Passing through:			Notes
	Lock	Pass	Lock and pass	
1	56	35	91	a For currents of 4 knots or more, the combined capacity of pass and lock exceeds channel capacity, so the latter controls.
2	57	59	116	
3	59	93	152	
4	61	174	214a	

Capacity of Sea-Level Canal with Tidal Regulation.—The 24-hour daily capacity of the sea-level canal with tidal regulation as assumed in these studies would be the combined capacities of the pass and lock—or 174 vessels of all sizes (86 vessels through the tidal lock and 88 vessels through the navigable pass).

Because there are no anchorage areas above the tidal lock or navigable pass, they would not be used when the channel is fogbound, and the capacity of the canal would be reduced accordingly. Observations made prior to the formation of Gatun Lake indicate that fogs in a sea-level canal would have an average duration of 6.6 hours and would reduce the operating day from 24 hours to 17.4 hours and the daily capacity of a controlled sea-level canal from 174 vessels to 126 vessels. In a 16-hour operating day, with operations suspended during the hours in which fog usually occurs, the daily capacity would be as shown in Table 13. It can be noted that, even with the most conservative

control of tidal currents, the capacity of a sea-level canal is in excess of the 69-vessel daily capacity required for a Panama Canal in the year 2000.

The average transit time in a controlled sea-level canal would be either 4 hours or 4.75 hours, depending on whether the pass or the lock was used.

Capacity of Sea-Level Canal Without Tidal Regulation.—If operating experience should eventually indicate that tidal regulation is unnecessary or if, during wartime, the regulating structures were not utilized or were rendered inoperative, the canal would have no structures on which the transits of vessels would be dependent. Its capacity would therefore be the capacity of the channel, which is dependent solely on vessel speed and spacing and on the length of the operating day as established by the occurrence of fog.

Using an average water speed of 10 knots and a vessel spacing of 2 miles established for the 4.5-knot current resulting from a 20-ft tide (Table 11), the daily capacity of the canal would be 240 vessels longer than 300 ft. Vessels longer than 300 ft are expected to comprise only 75% of all vessels transiting the canal (Table 9), and therefore 80 additional vessels shorter than 300 ft could be interspersed with the larger vessels; the daily capacity would then be 320 vessels.

Fog of average duration would reduce the operating day from 24 hours to 17.4 hours and the daily capacity from 320 vessels to 232 vessels. If the canal, because of fog or for other reasons, is operated 16 hours a day, the daily capacity would be 214 vessels—far in excess of the requirements for the year 2000.

At an average water speed of 10 knots, the time required to transit the sea-level canal without tidal regulation would vary from 3.3 hours to 5.0 hours.

SUMMARY

The present Panama Canal does not have sufficient capacity to prevent undesirable delays to traffic on design peak days beginning about 1960. With modification of the existing locks to eliminate extensive outages during overhaul periods and with tie-up stations provided in Gaillard Cut to reduce the delays created by fogs, the present lock canal would be adequate until close to the end of the century. In addition to these improvements, if navigation aids were provided which would permit navigation in fogs, the adequacy of the present canal would be further extended. This minimum plan of improvement, however, would not provide for the transit of vessels larger than those accommodated in the present locks.

The Panama sea-level canal or a completely modernized Panama lock canal with new and larger locks would have capacities greatly in excess of requirements for the year 2000. The modernized lock canal and the tidal lock of a sea-level canal would be able to transit any commercial vessel expected to be operating during the twentieth century. They would also permit the transit of much larger naval vessels than now exist. The navigable pass of the sea-level canal would permit the transit of any type of vessel of any size likely to be built prior to the year 2000.

The capacity of a lock canal is fixed by the physical dimensions and characteristics of its locks. In comparison, a sea-level canal has no such limitations, provides faster transit than does a lock canal, and has a larger reserve of capacity which can accept the short-time traffic peaks that may occur during wars.

FLOOD CONTROL

BY F. S. BROWN,[13] ASSOC. M. ASCE

SYNOPSIS

The development of a Panama sea-level canal would require that shipping be protected by adequate control of floods on the major rivers and streams tributary to the canal. If large uncontrolled flows entered the canal, the maximum velocities would be appreciably increased over those normally caused by tides, and dangerous crosscurrents would be generated at each point where large flood flows entered the canal. This paper defines the requirements for flood control in the proposed sea-level canal, describes the hydrological characteristics of the drainage basin, formulates the preliminary basis for hydraulic design of the flood-control projects, and develops the plan of control that has been adopted as an essential feature of the sea-level canal, wherein 87% of the tributary area would be diverted directly to the Atlantic Ocean and an additional 4% would be controlled by retarding reservoirs.

REQUIREMENTS FOR FLOOD CONTROL

Floods have rarely interfered with navigation in the present lock canal because all major tributaries of the watershed, except the upper Chagres River, enter Gatun Lake at remote distances from the navigation lane. Gatun Lake is impounded by a large earth dam 8 miles above the mouth of the Chagres River and has an area of 164 sq miles. The upper Chagres River enters the canal at Gamboa where the lake is narrow. Prior to the construction of Madden Dam in 1934, floods entering the canal at Gamboa twice required the temporary suspension of traffic. The control afforded by Madden Lake has largely ended the likelihood of futher major flood disturbances in the present canal.

Gatun Lake would be drained upon final conversion to a sea-level canal, and the rivers of the watershed, if uncontrolled, would enter the sea-level canal directly. The total area of the watershed (Fig. 13) would be 1,358 sq miles, 94% of which would lie on the Atlantic side of the Continental Divide. The largest tributary would be the Chagres River above Gamboa, with a drainage area of 520 sq miles, of which 393 sq miles are controlled by Madden Dam. Other important tributaries would be the Gatun River, draining 150 sq miles on the east side of the canal, and the Caño Quebrado and Trinidad rivers on the west side, with drainage areas of 120 sq miles and 313 sq miles, respectively. The watershed on the Pacific side of the Continental Divide is only 79 sq miles.

Combined Flood and Tide Flow in a Sea-Level Canal.—Tidal ranges of 20 ft in the Pacific and 2 ft in the Atlantic, which closely approximate maximum tide conditions, would produce a velocity of 4.5 knots near the Atlantic end of

[13] Chf. of General Eng. Branch, Special Eng. Div., The Panama Canal, Diablo Heights, Canal Zone.
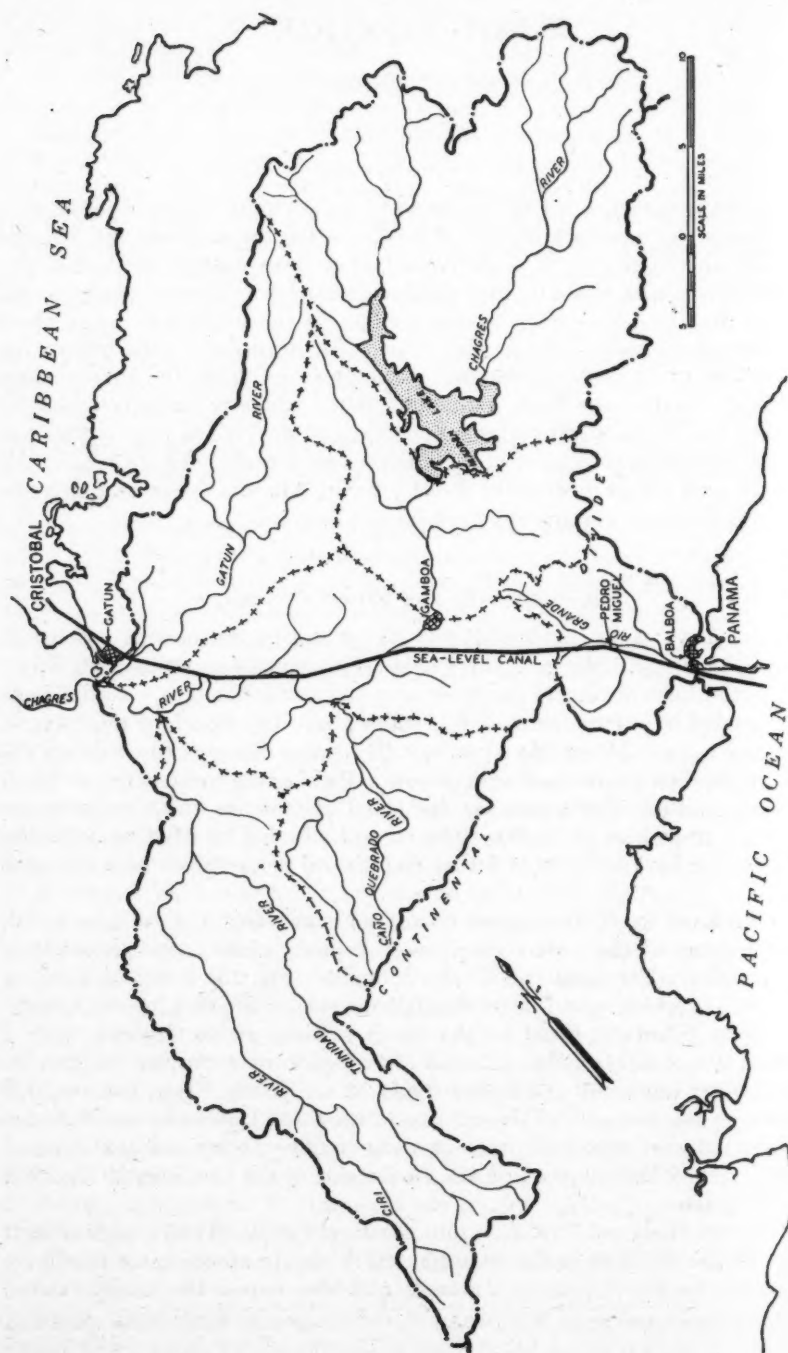
Fig. 13.—Watershed, Sea-Level Canal.

an uncontrolled canal. Ships of adequate power and good controllability could safely transit the canal in currents of 4.5 knots, but any appreciable increase in velocity would be undesirable. Tidal-regulating structures would be installed at the Pacific end of the sea-level canal for the reduction of tidal currents to limits probably as low as 2 knots in the initial period of canal operation. Flood control would be needed regardless of the extent of tidal regulation, although disturbances to shipping from floods would be more pronounced in an unregulated canal, a condition that may be imposed during wartime if the tidal-regulating structures were destroyed.

Fig. 13.—Watershed, Sea-Level Canal

(a) FLOOD INFLOW

(b) VELOCITIES

(c) WATER SURFACE PROFILES
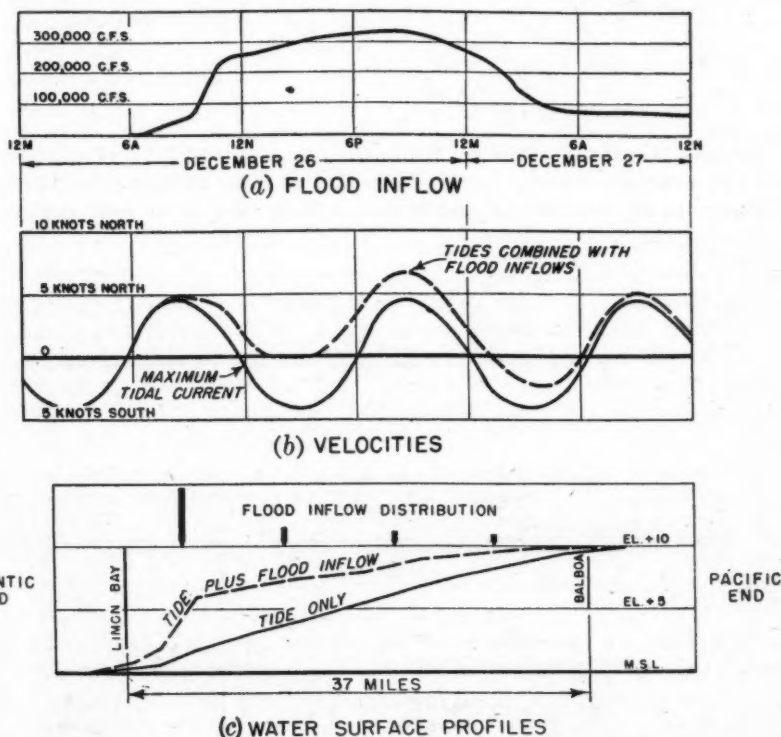
Fig. 14.—Velocities Observed in Model Tests to Determine Effects of 1909 Flood

The effect of large floods on velocities in an uncontrolled canal has been demonstrated in the hydraulic model of the sea-level canal by superimposing the flood of December 26–30, 1909, upon the conditions producing maximum tidal flow. This flood, largest of record on the Chagres River at Alhajuela, was extended with suitable adjustments to cover the entire watershed. The net rates of flood inflow into the canal were determined for two degrees of control: Case I, with only Madden Lake in operation (393 sq miles controlled), and case II, with a comprehensive flood-control system in operation (1,240 sq miles controlled). Fig. 14 summarizes the test results for case I and in-

cludes a comparison with tide flow only. Flows were introduced in the canal
at four locations corresponding to the main tributaries, those entering the
canal at Gamboa being modified to reflect the control afforded by Madden
Dam. The tests were run without tidal regulation. The profiles of Fig. 14
demonstrate that the gradient of flow is materially flattened upstream from
the point of largest inflow. This flattening tends to retard the entrance of
tidal flow into the canal and, as a consequence, the combination of tide and
flood flow would be less than the arithmetic sum of the separate flows. Maxi-
mum combined velocities observed during these tests (open canal; 20-ft tide)
were as follows:

Condition	Velocities (knots)
Case I, flood control.............................	7.0
Case II, flood control........	5.0
No flood inflow.............................	4.5

The high velocity of combined flow given by case I and the virtual restoration
of the combined velocity in case II to the value for natural tidal flow are in-
dicative of the requirement and effect of a large measure of flood control.

(a) PLAN

MIXED FRESH AND SALT WATER

SALT WATER

(b) SECTION

0   50 100%

(c) SALINITY AT X-X

-1 0 +1 2 KNOTS

(d) VELOCITY AT X-X
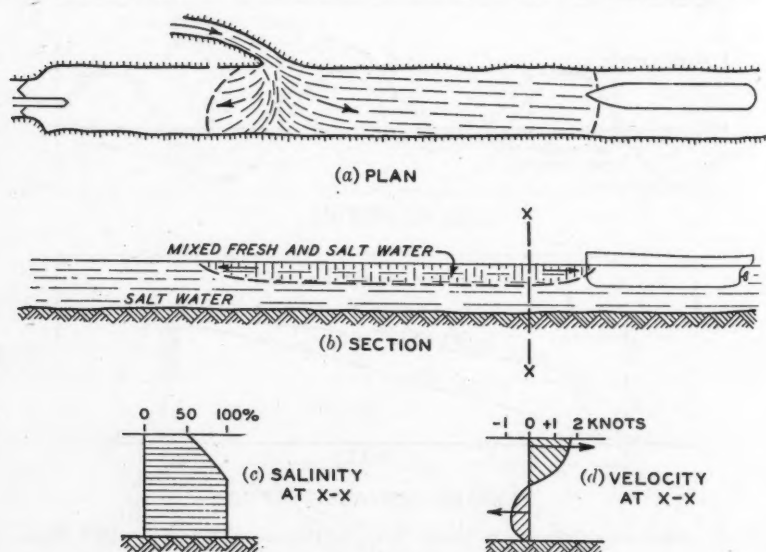
Fig. 15.—Inflow Tests, Existing Panama Canal

Density Currents in the Sea-Level Canal.—The foregoing tests were per-
formed using fresh water for both flood and tidal flow and hence do not fully
represent the conditions that would arise in nature. From observations in
tidal estuaries, it is known that fresh-water and salt-water flows tend to retain
separate identity, the lighter fresh water, of course, remaining on top of the salt
water. Recent full-scale measurements have been made of fresh water from a
tributary stream flowing into the salt water of the sea-level approach channel to
the Miraflores Locks. The results of these field tests are summarized in Fig. 15.

Fig. 15(*a*) indicates the pattern of currents near the surface, and the movement of the fresh-water front as it spreads out over the salt water, The successive locations of this front were observed with electrical salinity meters.

The water near the surface of the canal was not wholly fresh water but a mixture of fresh and salt, starting with about 50% salinity at the top and increasing to 100% salinity at about half the channel depth. The water moved in one direction near the surface and in the opposite direction near the bottom. For a tributary inflow of 24,000 cu ft per sec, the velocities in the canal averaged 1.8 knots in the upper zone—or approximately three times the velocities that would have occurred had the discharge over the full cross section been uniform.

Summary of Requirements.—It is difficult to define specific criteria for flood control, since the necessity for control is measured by the safety and efficiency of navigation in the canal, the standards for which are largely qualitative As a result of the tests and studies performed on flood currents in the canal, however, little doubt exists that the plan of control should be comprehensive, approaching virtual elimination of flood disturbances.

The Drainage Basin

Topography.—The principal subdivisions of the Panama Canal drainage area are illustrated in Fig. 13. The watershed is characterized by conical hills of relatively low relief in the vicinity of the canal, changing to irregular knife-like ridges in the upper regions. Peak elevations of from 500 ft to 1,000 ft above sea level near the canal increase to 2,000 ft in the Gatun River basin, 2,500 ft in the upper watershed of the Chagres River, and 3,500 ft in the headwaters of the Trinidad River. The general steepness of the watersheds produces a rapid concentration of runoff.

Surface Cover.—In the thickly-forested areas of the upper regions, dense foilage prevents sunlight from reaching the ground, and consequently the ground cover is sparse and offers little deterrent to rapid runoff. In regions of lower elevation, the forest cover is less dense, the undergrowth is thick, and open areas are covered with a rank growth of grass, causing rainfall losses and the retardation of runoff to be more pronounced.

Climate.—The Canal Zone region has a distinct seasonal variation of rainfall, a uniform air temperature, and a high relative humidity. The dry season generally extends from January through April, and the wet season spans the remaining 8-month period from May through December. The mean monthly air temperature in the Canal Zone varies only from 2° F to to 3° F for the entire year, and the range between extreme temperature is less than 40° F. The highest recorded temperature is 98° F and the lowest is 59° F, both occurring at Madden Dam.

Rainfall.—The mean monthly rainfall at the Atlantic and the Pacific ends of the canal is shown in Fig. 16. Approximately 92% of the mean annual rainfall occurs during the wet season, and 16% of the annual rainfall occurs in November. All major floods in the Canal Zone region have occurred in the last 3 months of the year.

Fig. 17 is an isohyetal map of the mean annual rainfall on the canal drainage area. Note the gradation of rainfall across the Isthmus, ranging from depths
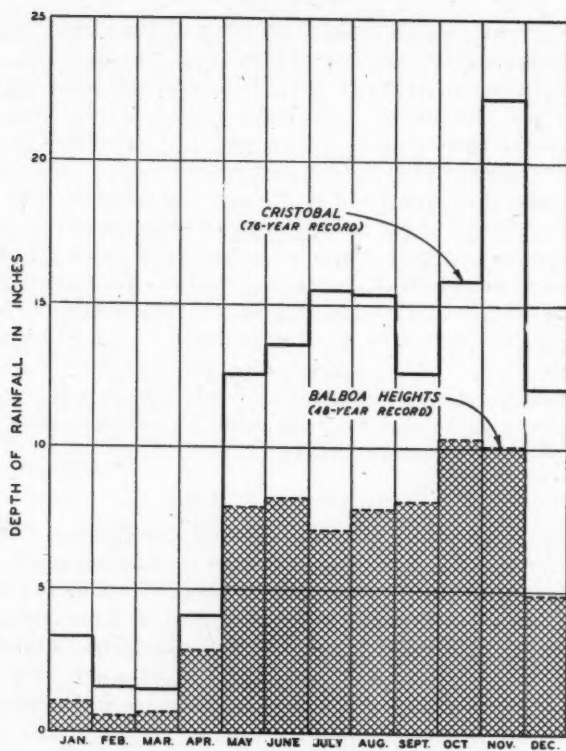
Fig. 16.—Mean Monthly Rainfall
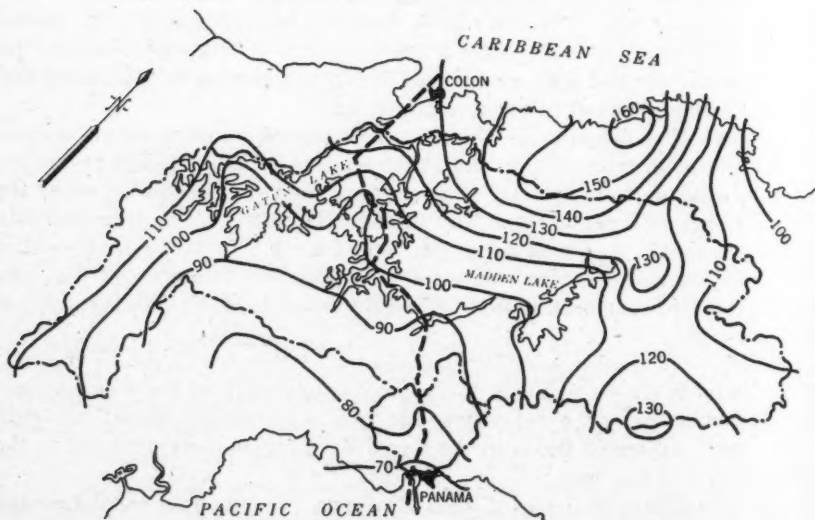
Fig. 17.—Isohyetal Map of Mean Annual Rainfall

of 160 in. on the Atlantic coast to 70 in. on the Pacific coast. This gradation is also characteristic of monthly rainfall and occurs during many of the major flood-producing storms.

A large part of the rainfall results from convective activity in which the areal distribution is limited, the duration is a few hours, and the intensity is relatively high. Large flood-producing storms result from frontal activity and generally extend over the entire watershed of the canal. Rainfall caused by frontal activity usually has a lower intensity but may continue intermittently for a week or longer in storm periods having a succession of frontal passages. Maximum point rainfall in the Canal Zone region and at several locations in the United States is shown in Table 14.

TABLE 14.—MAXIMUM POINT RAINFALL; CANAL ZONE REGION
AND UNITED STATES

(Rainfall Depth in Inches)

Duration	Canal Zone and vicinity	FIRST ORDER WEATHER BUREAU STATIONS				
		Los Angeles, Calif.	New Orleans, La.	Key West, Fla.	St. Louis, Mo.	Washington, D. C.
5 min.........	0.90	0.44	0.77	0.65	0.60	0.80
10 min........	1.76	0.66	1.20	1.03	1.04	1.21
1 hour........	5.68	1.51	3.66	4.30	3.47	3.42
24 hours......	13.62	7.36	14.01	13.54	8.78	7.31
Annual.......	237.3	40.3	85.7	58.5	68.8	61.3

Rainfall and Discharge Records.—Heterogeneous rainfall and stream-flow records are available to the hydrologic investigator, some dating back to the 1860's. The primary objective of past observations has been the collection of data for definition of the dependable supply of water for lockages rather than for the development of flood-control projects. Although the number and distribution of rainfall stations in operation in recent years, particularly since 1941, have been sufficient to approximate a pattern of the mean monthly and mean annual rainfalls on the watershed, the records are insufficient for accurate study of storm and flood relations in any tributary area. The best records of discharge are those of the upper Chagres River where observations were started at Alhajuela in 1899 and have been continued to the present time, although the construction of Madden Dam in 1943, immediately upstream, prevented further recording of natural flood discharges at this station. Five rainfall stations have been in operation above Alhajuela since the early 1930's and three more were added in 1941. The general inadequacy of the records has made it necessary to develop data for the hydraulic design of the various projects largely from the Chagres River records.

Comparative Distribution of Rainfall.—Table 15 shows the comparative distribution of rainfall on the major drainage areas of the watershed. The comparison was made for four periods—the annual; the combined flood season months of October, November, and December; the month of November alone; and the average of twenty-two storms. For easy comparison, the values for the Chagres River above Madden Dam have been equated to 100% in each

column.   This study was undertaken to develop adjustment factors for transfer of storm and flood data from the Chagres River above Madden Dam to other areas.   The values for the mean November rainfall were selected because most large floods have occurred during that month and because the November ratios would produce a larger design flood on the other areas.

TABLE 15.—AREAL DISTRIBUTION OF RAINFALL
(Rainfall Depth in Inches)

Drainage area	MEAN ANNUAL RAINFALL		MEAN FLOOD SEASON RAINFALL (OCTOBER-NOVEMBER-DECEMBER)		MEAN NOVEMBER RAINFALL		AVERAGE RAINFALL OF 22 STORMS (1941–1946)	
	Average depth	%	Average depth	%	Average depth	%	Average depth	%
Chagres River above Madden Dam......	116.0	100	45.0	100	15.9	100	4.2	100
Chagres River, Madden Dam to Gamboa..	100.9	87	36.7	82	14.8	93	3.4	81
Gatun River..............................	131.0	113	51.5	114	22.4	141	4.9	117
Caño Quebrado River....................	90.4	78	34.0	76	14.5	91	3.2	76
Trinidad River..........................	103.7	89	41.4	92	14.6	92	3.1	74
Entire Watershed.......................	109.0	94	42.3	94	16.0	101	3.8	90

Major Storms and Floods on the Upper Charges River.—Since the beginning of authentic records in 1899, the greatest peak discharge of the Chagres River at Alhajuela was 140,000 cu ft per sec (356 cu ft per sec per sq mile), recorded during the storm of December 26–30, 1909.   The second largest peak discharge was 129,000 cu ft per sec (328 cu ft per sec per sq mile), recorded at Alhajuela on December 3, 1906.

The largest volume of runoff in a single flood period was produced during the storm of November 12–23, 1935, and averaged 36.8 in. above Alhajuela. Summary data of the floods described are given in Table 16.

TABLE 16.—MAJOR FLOODS OF CHAGRES RIVER AT ALHAJUELA
(393 Sq Miles)

Date of flood	RAINFALL		PEAK DISCHARGE		Meyer[a] rating	24-HOUR VOLUME	
	Period (days)	Average depth	Cu ft per sec	Cu ft per sec per sq mile		Day-sec-ft	In.
December 26–30, 1909..	5	14.8[b]	140,000	356	71	94,600	9.0
December 3, 1906.....	1	5.2[b]	129,000	328	65	77,800	7.4
November 12–23, 1935.	12	36.8	....[c]	....[c]	....[c]	52,200	5.0

[a] Expressed in percentages, in which 100% equals 10,000 times the square root of the drainage area.   [b] Rainfall observed at Alhajuela.   [c] Construction of Madden Dam in 1934 prevented further recording of peak discharge.

Flood Frequency.—The occurrence of Chagres River floods since the beginning of records in 1899 is plotted with respect to the magnitude of peak discharges and 24-hour discharges in Fig. 18.   Computations of occurrences have

been made by arranging the floods in decreasing order of magnitude and dividing the rank of the flood into the years of record.   This assigns a frequency to the maximum value equal to the period of record.

Rainfall-Runoff Relations.—In examining past records, it was noted that the recorded runoff was frequently much more, and occasionally less, than the
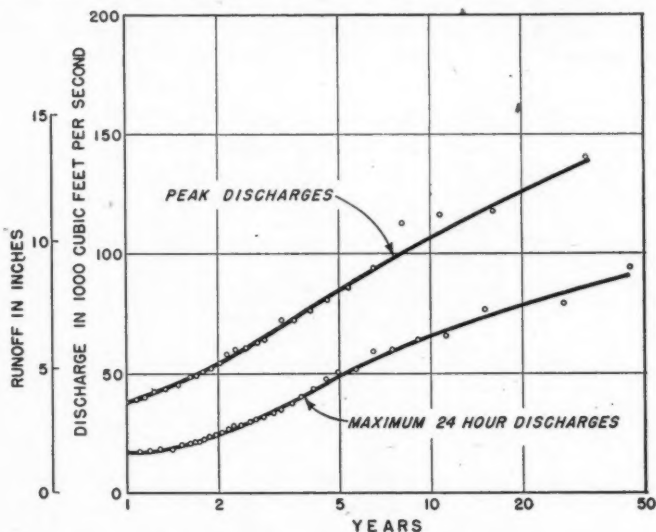
FIG. 18.—FLOOD OCCURRENCES, CHAGRES RIVER AT ALHAJUELA

corresponding recorded rainfall, presumably the result of an insufficient number of stations.   As a consequence, rainfall losses were not directly determinable from observed data, and large adjustments of observed rainfall were frequently required for a satisfactory reproduction of recorded flood hydrographs in unit hydrograph verification studies.

PROJECT DESIGN FLOOD

Hydrographs of two extended flood periods yielding the largest volumes of runoff on the Chagres River at Alhajuela are shown in Fig. 19.   These two floods were studied for the purpose of developing a project design flood that could be used for selecting the storage capacities of reservoirs and the discharge capacities of outlets and diversion channels.   The total period of the plotting is 50 days.   The 1909 period has three distinct peaks, the last corresponding to the maximum of record cited in Table 16.   The 1935 period has two distinct peaks, the first peak corresponding to the 12-day flood of Table 16.   The accumulative runoff for the two flood periods is of the same order of magnitude, the highest being 75 in.   Because of the relatively short period of record and the importance of positive elimination of flood interference in the canal, the volume of each of these floods was increased 25%.   Routing of these floods through the Madden Lake reservoir demonstrated that the maximum storage requirements would result from the augmented 1935 flood.   This

flood therefore was adopted as the project design flood for the existing Madden Lake and as the general basis for design of projects on other large areas. The flood from the Madden Lake area to other areas was transposed by applying two ratios: (1) The direct ratio of the drainage areas; and (2) the ratio of the mean November rainfall of Table 15.
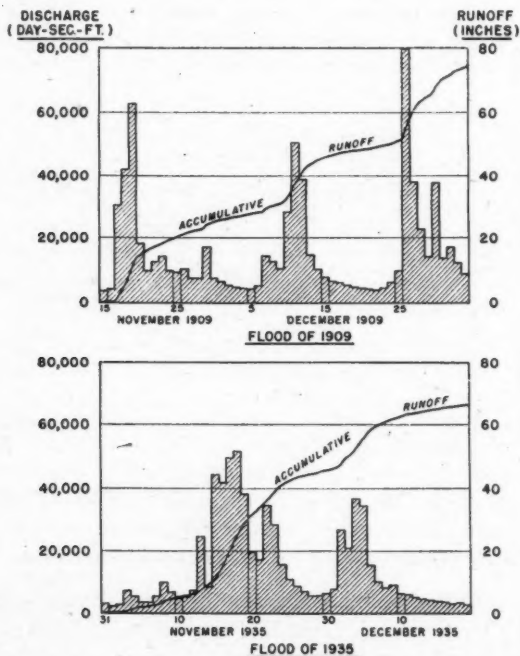
FIG. 19.—FLOOD HYDROGRAPHS, CHAGRES RIVER AT ALHAJUELA

Project Design Flood for Small Drainage Areas.—The proposed flood-control system includes the regulation of runoff on several small tributary areas of from 10 sq miles to 30 sq miles. To assure dependable control on such areas, the project design flood was selected to be 75% of the spillway design flood—a flood of shorter duration but higher discharge than the transposed flood of November, 1935.

SPILLWAY DESIGN FLOOD

The spillway design flood determines the size of spillways and the maximum water level in the reservoir and is derived from the estimated maximum possible storm for the watershed. This storm was established for the Madden Dam areas by the United States Weather Bureau in 1942 following an investigation of all the measurable meteorologic characteristics of the past storms that have produced heavy rainfall over or near the Canal Zone region. Depth-duration data of the storm for the Madden Dam area, extrapolated to apply to areas ranging from 10 sq miles to 600 sq miles, are shown in Fig. 20.

Development of the spillway design flood on the various watersheds required (1) selection of applicable rainfall data from Fig. 20, (2) adjustment of selected rainfall using the ratio of mean November rainfall between the various large areas and the Madden Lake drainage area to permit transposition of data, and (3) conversion of adjusted rainfall into flood runoff using inflow unit hydrographs. On the small drainage areas of 30 sq miles or less, the rainfall values of Fig. 20 were used without adjustment since these values could result from an intense convective storm of short duration which would be of like magnitude in any region.
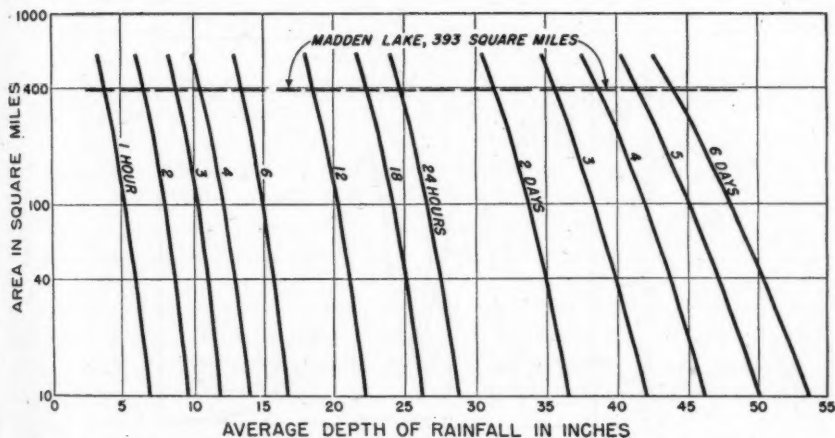
FIG. 20.—DURATION-DEPTH-AREA CURVES OF MAXIMUM POSSIBLE STORM, MADDEN LAKE

Inflow Unit Hydrographs for Reservoir Watersheds.—Unit hydrographs for converting storm rainfall into spillway design flood inflow for each flood-control project were constructed synthetically by the Snyder method.[14] General guidance for appropriate values of the empirical constants C_t and C_p, which

TABLE 17.—SNYDER'S CONSTANTS FOR ADOPTED UNIT HYDROGRAPHS, UPPER CHAGRES RIVER BASIN

Station	Stream	Drainage area (sq miles)	Unit time of rain (hours)	C_t	$\frac{640}{C_p}$	Peak discharge, q_p (cu ft per sec per sq mile)
Alhajuela	Chagres	393	2	1.00	400	55
Chico	Chagres	160	1	0.40	500	192
Candelaria	Pequeni	52	1	0.45	600	290
Peluca	Boqueron	35	1	0.50	400	200
For general use	....	10–200	1	0.50	500	....

reflect the lag time and peaking characteristics of the watershed, respectively, was obtained by constructing unit hydrographs for four gaged areas above Alhajuela and verifying them by the reconstruction of twelve floods. The constants derived from the study are given in Table 17, and a typical example

[14] "Synthetic Unit Graphs," by Franklin F. Snyder, *Transactions*, Am. Geophysical Union, Vol. 19, Pt. I, 1938, p. 447.

of verification is shown in Fig. 21. The final inflow unit hydrograph constructed for Madden Lake, using constants summarized in Table 17, was verified by the reconstruction of a number of floods which have occurred on the watershed since the construction of the dam in 1934. An initial loss of

FLOOD OF NOVEMBER 6-10, 1931
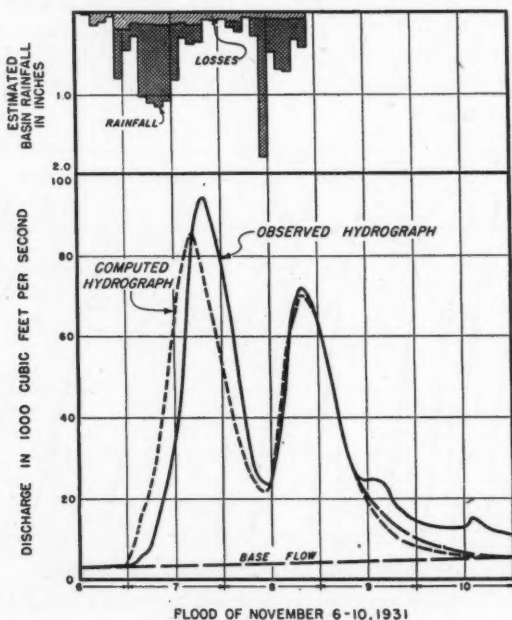
Fig. 21.—Unit Hydrograph Verification, Chagres River at Alhajuela, 1909 Flood

0.5 in. and infiltration at the rate of 0.05 in. per hr were deducted from rainfall in these studies, and a base flow of 10 cu ft per sec per sq mile was added to the computed inflow to complete the spillway design flood hydrograph.

The Flood-Control Plan

Early French and American Plans.—The importance of controlling floods on the tributaries to a Panama sea-level canal was appreciated by both the French and American proponents of a sea-level canal. The French envisioned a dam at Gamboa for regulation of inflow into the canal from the Chagres River and diversion of the Gatun and Trinidad rivers in separate channels leading to the Caribbean Sea. In 1906, the majority group of the American Board of Consulting Engineers recommended the construction of a sea-level canal and proposed the following flood-control plan, similar to that of the early French planners:

"To control the Chagres River, a dam * * * is proposed at Gamboa * * * forming a reservoir called Gamboa Lake, of which the maximum flow line is to be at elevation 170 * * *.

"This dam is to be fitted with controlling sluices by which a maximum discharge of 15,000 cubic feet per second is to be admitted to the canal prism * * *. Of the tributaries entering the Chagres below Gamboa, the most important are diverted entirely from the canal and conducted by separate channels to the sea * * *.

    *   *   *

"The Caño [Quebrado] and the Gigante [now included as part of the Caño Quebrado], * * * are to be cut off by dams. The Trinidad will occupy the old channel of the Chagres River and the Chagres diversion. The Gatun will be cut off from the canal by the partly finished Gatun diversion, * * *."

Development of Present Plan.—The physical possibilities for developing flood control for a Panama sea-level canal are abundant and variable. Control could be established by a system of reservoirs located in the lower central part of the main drainage basins, similar to the location of Madden Lake on the Chagres River Valley above Gamboa. Inflow into the canal from such a system would be composed of regulated flood releases from the reservoirs and unregulated runoff from areas below the reservoirs. A study of stream-flow records on the Upper Chagres River indicated that this system of control would permit a peak discharge of 45,000 cu ft per sec to enter the sea-level canal at Gamboa on the average of once in 5 years. This rate of inflow is considered to be from two to three times greater than the maximum that could be tolerated without seriously disturbing navigation. It was concluded that an effective plan of control by reservoirs would require location of dams close to the sea-level canal to prevent runoff from areas of even relatively small size from entering the canal directly.

Diversion of runoff from tributary areas, particularly for the larger streams, would obviously afford the most satisfactory method of preventing floods from interfering with navigation. The large drainage areas lie on the north or Atlantic side of the Continental Divide and, if uncontrolled, would discharge into the canal at distributed locations from Gamboa to Gatun (Fig. 13).

The two main tributaries on the west side, the Caño Quebrado and the Trinidad rivers, could be diverted by excavating a relatively shallow channel in the broad valley of the old Chagres basin, generally paralleling the sea-level canal and connecting with the former channel of the Chagres River below Gatun Dam. Another west side diversion plan would employ material available from sea-level canal excavation for the formation of a series of low dams between west side islands in Gatun Lake to create a diversion reservoir of moderate depth in which a permanent pool could be maintained for the general benefit of better sanitation and access in the region. The material from canal excavation would be available for embankment construction at little or no extra cost because large amounts would be barged to the lake for economical disposal regardless of flood-control requirements.

Control of the two principal tributaries on the east side, the Chagres and Gatun rivers, could be effected generally by either of two plans: (1) By diverting both tributaries, or (2) by diverting Gatun River to the sea and controlling the Chagres River by construction of a regulating reservoir at Gamboa as proposed in the French and American plans. Complete diversion would in-

TABLE 18.—Pertinent Data

Line	Reservoir	River	Drainage area (sq miles)	Elevations, Pacific Level Datum (Ft)					Peak outflow[a] (cu ft per sec)
				Normal water surface	Spillway crest	Top of dam	Design Flood		
							Project	Spillway	
(1)		(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	Madden (existing)..	Chagres	393	210	250[b]	273	239.8	262.9	30,000
2	Gamboa...........	Chagres	127	95	140	174	129.0	158.8	38,200
3	Monte Lirio.......	Gatun	180	55	55	92	61.5	82.2	72,000
4	Trinidad..........	Trinidad and Caño Quebrado	488	55	55	82	58.4	71.3	58,000
5	Madinga..........	Mandinga	10.5	150	208	235	207.2	223.7	2,970
6	Cocoli............	Cocoli	11.5	70	107	135	106.6	122.4	3,130
7	Miraflores........	Five small rivers	30.0	70	90	115	89.8	102.3	5,890

[a] Peak outflow, project design flood, in cubic feet per second

clude a reservoir at Gamboa, a 13.7-mile diversion channel from Gamboa north-ward to the Gatun River Valley, and a second diversion channel 4.7 miles long from the north rim of the Gatun Valley to the sea. This plan is similar to the one proposed by John G. Claybourn,[7] M. ASCE. Under plan (2), the 13.7-mile diversion channel would not be constructed and the capacity of the reservoir on the Chagres River at Gamboa would be increased to reduce further the reservoir outflow, which in this case would enter the sea-level canal. In either system, Madden Lake would be operated largely for flood control of the upper Chagres River. The plan of complete diversion is considered decidedly superior because it would avoid a prolonged inflow of regulated releases at Gamboa during the wettest months. It would be difficult to reduce these releases sufficiently to prevent possible disturbances from crosscurrents or eddies formed by the interflowing of masses of salt water and fresh water.

The Adopted Flood-Control Plan.—The adopted plan of flood control embodies (*a*) complete diversion of all runoff from the major tributaries, and (*b*) control by regulating reservoirs on several small streams entering the canal south of Gamboa. The plan is comprehensive, since less than 9% of the entire watershed would remain uncontrolled. The largest uncontrolled area would not exceed 4.5 sq miles. The location of the flood-control projects and a summary of the areas controlled are shown in Fig. 11. The diversion of major streams lying on the Atlantic side of the Continental Divide and draining 87% of the watershed is accomplished by two distinct projects designated the East Diversion and the West Diversion. General descriptions of the projects and their operation are outlined in the paragraphs following, and statistical data pertaining to the reservoirs and dams are given in Table 18.

East Diversion

Madden Lake.—The storage in Madden Lake[15] now used principally for power generation and lock water supply would be reallocated, approximately

[15] "More Water for the Panama Canal," by E. S. Randolph, *Civil Engineering*, May, 1932, pp. 283–288.

—FLOOD–CONTROL RESERVOIRS

Available Storage Below Spillway Crest		Embankment Dimensions (Ft)			Uncontrolled length of spillway (ft)	Gated Outlets		Ungated Outlets		Line
Acre-ft	In.	Crest length	Crest width	Crest height		No.	Size (ft)	No.	Size (ft)	
(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	
382,000[b]	18.2[b]	5,700	22	183	(4–100)[c]	6	5.67 by 10	2	7-ft needle valves	1
357,000	52.7	3,900	100	129	1,000	1	Diameter, 38 ft	1	Diameter, 45 ft	2
..	..	23,000	100	82	600	5	20 by 20	..	......	3
..	..	47,000	50	72	500	3	20 by 20	..	......	4
6,000	10.7	1,200	50	117	100	1	Diameter, 10 ft	1	Diameter, 9 ft	5
6,820	11.1	1,000	50	95	100	1	Diameter, 11 ft	1	Diameter, 10 ft	6
18,500	11.6	6,500	50	85	200	2	Diameter, 11 ft	1	Diameter, 10 ft	7

[b] Top of 18-ft crest gates.   [c] Four, 100-ft crest gates.

60% being assigned to flood control and 40% to power. The flood-control storage would be equivalent to a runoff depth of 18.1 in. on the 393 sq miles of tributary watershed. Regulated discharges up to a maximum of 30,000 cu ft per sec would be released from Madden Dam. The project design flood would utilize two thirds of the assigned flood-control storage, the remainder constituting a reserve for such contingencies as silting and possible changes in operating procedure.

Gamboa Reservoir.—This reservoir would receive discharges from Madden Lake and runoff from the 127 sq miles of drainage area below Madden Dam. The normal level of the reservoir would be El. 95, 10 ft above the present Gamboa arm of Gatun Lake. The project design flood would raise the water level in Gamboa reservoir to El. 129, leaving approximately one third of the storage below El. 140.0, the crest of the spillway, available as a reserve for operating contingencies. Outflow from this reservoir would be diverted northward via the Chagres River diversion channel to the Monte Lirio reservoir, third and last reservoir in the East Diversion. Any discharge that might occur over the Gamboa spillway would enter the abandoned channel of the existing canal and would flow into the sea-level canal. The top of the Gamboa Dam, an earth and rock embankment (Fig. 22) would be 125 ft above the river bed and the crest length would be 1,800 ft. A 38-ft-diameter tunnel would be constructed through the south abutment for diversion of the Chagres River during construction of the dam and would be retained for emergency release of flood discharges into the sea-level canal under remote contingencies such as closure of the Chagres diversion channel by slides.

Chagres Diversion Channel.—Diverted flows would leave the Gamboa reservoir through a 45-ft-diameter tunnel, 2,700 ft long, and thence through an open channel, 13.7 miles long, varying in width from 100 ft in the sections of deeper cut to 500 ft in the low areas (Fig. 23(b)). As shown in Fig. 23(c), two weirs would be constructed in the diversion channel—the upper, with crest at El. 95, would be located 6 miles below the tunnel outlet and would control

the normal level of the Gamboa reservoir; and the lower, with crest at El. 70, would be located 6 miles farther downstream. These weirs would keep the bottom of the channel submerged in the dry season and thus prevent inception of natural growth. Discharge from the Gamboa reservoir would be controlled
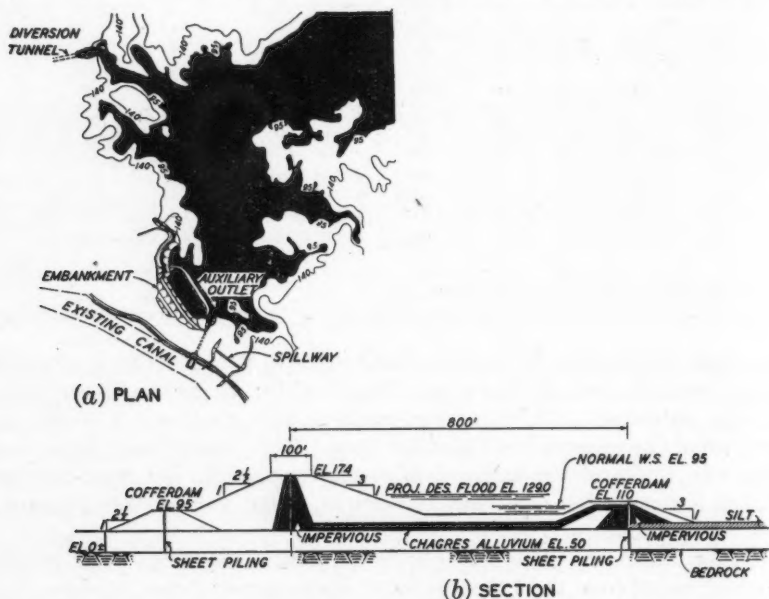
FIG. 22.—PLAN AND SECTION, GAMBOA DAM
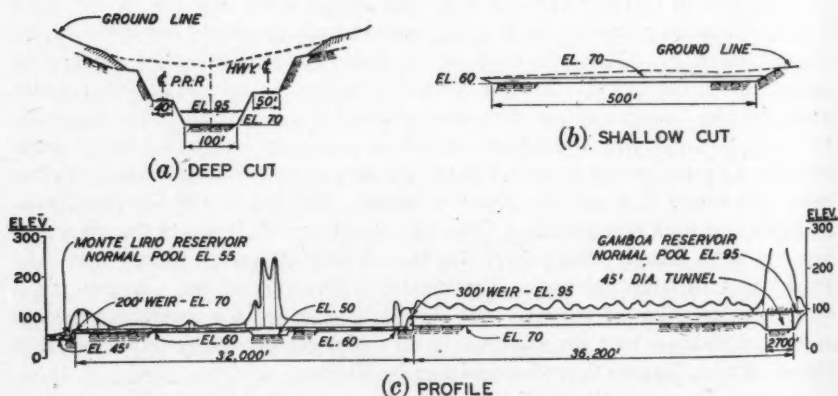
FIG. 23.—SECTIONS AND PROFILES, CHAGRES DIVERSION CHANNEL

by the natural capacity of the diversion channel and upper weir at low reservoir stages and by the capacity of the tunnel at higher stages. Valleys of existing streams crossed by the diversion channel would require nominal diking on the sea-level canal side.

Monte Lirio Reservoir.—Monte Lirio reservoir would occupy the lower valley of the Gatun River now inundated by the eastern arm of Gatun Lake and would receive flows diverted from the Gamboa reservoir, runoff from streams intercepted by the Chagres diversion channel, and flow from the Gatun River. The normal level of the reservoir would be maintained at El. 55, 30 ft below the present level of Gatun Lake, a level which is suitable for domestic water supply, area sanitation, and hydroelectric power development. The Monte Lirio Dam (Fig. 24(a)) would be formed by strengthening the existing Panama Railroad embankment which extends between several islands across the eastern part of Gatun Lake for a total distance of 4.4 miles.

All flows received in the Monte Lirio reservoir would be diverted to Las Minas Bay on the Caribbean coast through a channel 4.7 miles long (Fig. 24(b)). The diversion channel would have a width increasing from 200 ft at
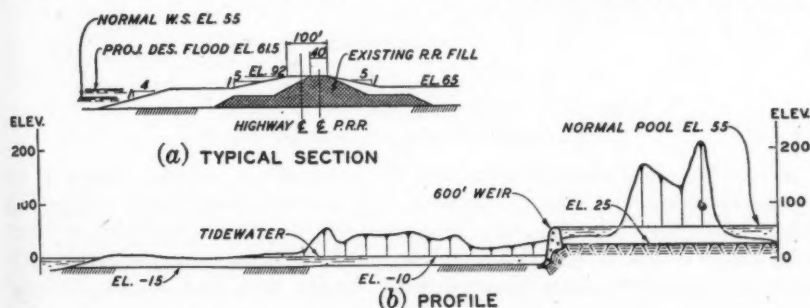
FIG. 24.—MONTE LIRIO DAM AND DIVERSION CHANNEL

the reservoir to 1,000 feet near Las Minas Bay. A 600-ft spillway weir with crest at El. 55 and an outlet structure containing five gates would be constructed in the channel 1 mile from the reservoir.

The outlet gates would be operated to maintain the water surface at El. 55 in the Monte Lirio reservoir and, during the passsing of a flood, would be fully open when the reservoir level rose above El. 55. The project design flood would raise the water surface of Monte Lirio reservoir to El. 61.5.

WEST DIVERSION

Two interconnected reservoirs, the Caño Quebrado reservoir and the larger Trinidad reservoir, with normal water surfaces at El. 55, would comprise the West Diversion (Fig. 11). Discharge from the Caño Quebrado reservoir would flow through an uncontrolled channel circling the south and west sides of Barro Colorado Island, and would join the Trinidad reservoir near the existing Gatun Dam. A 500-ft spillway with crest at El. 55 and an outlet structure containing three gates constructed in the west abutment of the existing Gatun Dam (Fig. 25(a)) would serve both reservoirs and would be regulated in the same manner as the Monte Lirio spillway and outlet structure. Discharges would flow to the sea through the old channel of the Chagres River. The project design flood would raise the water surface in the reservoirs to El. 58.4.

The dams for the West Diversion (Fig. 25(*b*)) would consist of low embankments placed on broad spoil areas of sea-level canal excavation. The surface of the spoil would be at El. 65 and the crests of the embankments would be at El. 82. The dams would extend from Gatun Dam to a point 4 miles north of Gamboa and would connect numerous islands in the present lake.

(*a*) UPSTREAM ELEVATION OF SPILLWAY

(*b*) TYPICAL SECTION
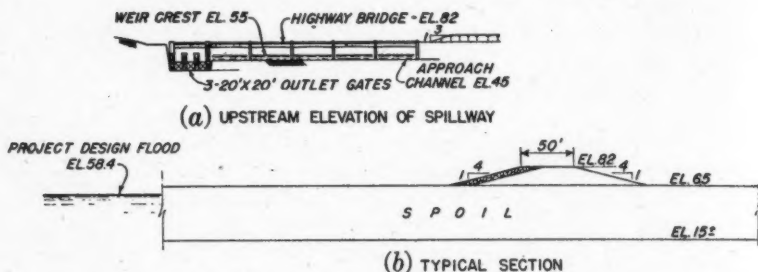
FIG. 25.—TRINIDAD AND CAÑO QUEBRADO DAMS

CONTROL OF SMALL TRIBUTARIES

Three retarding reservoirs on small tributary areas between Gamboa and Balboa Harbor (Fig. 11) are included in the flood-control plan. These are designated as the Mandinga, the Cocoli, and the Miraflores reservoirs, and control areas of 10.5 sq miles, 11.5 sq miles, and 30.0 sq miles, respectively. The last consists of five interconnected small basins with two outlet channels. Each reservoir would have a spillway and both uncontrolled and gated outlets. The maximum discharge into the canal at any one point would approximate 3,000 cu ft per sec.

FOUNDATIONS AND CONSTRUCTION MATERIALS

Preliminary explorations have revealed that sound rock is available at suitable depths in all foundation areas for spillway and outlet structures and in areas of tunnel excavation. Likewise, no major problems are anticipated in excavation of diversion channels. Most of the 13.7-mile diversion channel from Gamboa to Monte Lirio lies in regions of hard rocks of volcanic origin and medium-hard siltstones and limestones. The channel from the Monte Lirio reservoir to the sea would be located largely in sound siltstone and sandstone of the Gatun formation. The river section of the Gamboa Dam would be founded on alluvium, 50 ft thick, composed largely of sand and gravel. Measures for watertightness and safe relief of seepage would be incorporated in the design of the dam, as illustrated in Fig. 22(*b*).

Parts of the embankments for the Monte Lirio, Trinidad, and Caño Quebrado dams would be founded on a relatively soft alluvial deposit of organic clays, silts, and sands that is termed Atlantic muck. Fortunately, the great abundance of spoil material from the sea-level canal excavation would permit Caño Quebrado and Trinidad dams to be founded entirely upon a broad terrace of sound excavation spoil extending not less than 1,500 ft on both sides of the center line—with a top at El. 65, only 17 ft below the crest of the dams.

The Monte Lirio Dam, which would be an enlargement of the railroad causeway completed in 1912 across the eastern arm of Gatun Lake, would be similarly reinforced with broad berms for distribution of load.

Suitable and extensive deposits of gravel are available near the Gamboa dam site, and impervious, random, and rock-fill materials for the Gamboa Dam would be obtained from spillway excavation. Fill for embankments in the lake area would come mainly from canal excavation transported by barge.

CONSTRUCTION

Flood-control construction would be scheduled to utilize the material from canal excavation as it becomes available and to avoid unnecessary peaks in personnel. None of the structures would be required to function during the canal construction period, but Madden Lake would be assigned largely to flood control during the construction of the Gamboa Dam, the only project requiring major cofferdam construction and river diversion. The earth-fill cofferdams for the Gamboa Dam would form a part of the permanent structure.

OPERATION OF FLOOD–CONTROL PROJECTS
DURING FINAL LOWERING OF LAKES

All flood-control projects would be placed in operation prior to the rapid lowering of Gatun and Miraflores lakes upon conversion of the canal to sea level. More than 68% of the normal volume of Gatun Lake would be cut off by flood-control dams and thus the time of emptying the central region of the lake would be greatly accelerated. The total interruption to traffic at this time would be less than 7 days, including the time required for removing the lake-retaining barriers in the channel for the sea-level canal.

HYDROLECTRIC POWER

Hydroelectric power is generated in the Canal Zone at Gatun and Madden dams, which have an installed capacity of 46,000 kw. The Gatun station would be abandoned on conversion of the canal to sea level and, as noted previously, the storage assigned to power in Madden Lake would be reduced to one third of the total storage, sufficient to produce 13,000 kw at a 40% load factor. Opportunities would be available at the Monte Lirio and Trinidad spillway structures for small hydroelectric installations. The plant at Monte Lirio, receiving regulated flow from Madden Dam and unregulated runoff from the remaining watershed of 307 sq miles, would have a firm capacity of 13,500 kw at a 25% load factor. The Monte Lirio reservoir would not be regulated for power because drawdown of the normal water surface would affect the quality of water for domestic supply adversely. The Trinidad and Caño Quebrado reservoirs would be regulated between El. 55 and El. 45, permitting the generation of 7,500 kw of primary power at a load factor of 50%.

SAFETY OF STRUCTURES DURING WARTIME

The dams in the lake area would actually be low levees ranging from 17 ft to 27 feet in height constructed on broad spoil terraces at El. 65. These terraced areas would be virtually unbreachable by bombing of any type and a

breach in the levees would be of no consequence since the occurrence of a flood even as large as the project design flood would not raise the water level in either the East Diversion or the West Diversion above El. 65. If the spoil terraces were ever breached, the outlet conduits in the Trinidad and Monte Lirio spillway structures could be left open for continued diversion of flow to the sea with only minor repair of the breach. A breach in Madden Dam would not result in overtopping the Gamboa Dam even if both reservoirs were filled to spillway levels at the time—an extremely remote possibility. The Gamboa Dam would be difficult to breach by any conceivable conventional bomb, as it would be 350 ft wide at the level of the project design flood and 650 ft wide at normal water surface.

Summary

The control of floods on the tributaries of a Panama sea-level canal, in accordance with the adopted system, would completely eliminate hazards to shipping from flood inflows. The flood-control system would have sufficient capacity to control flows well in excess of the largest flood that has occurred in 47 years of record. Runoff from 87% of the area tributary to the sea-level canal would be diverted directly to the Atlantic Ocean and 4% would be controlled by retarding reservoirs. It would be difficult to breach the flood-control dams adjacent to the canal in wartime. If breached, only minor remedial work would be needed to restore the system to effective operation.

TIDAL CURRENTS

BY J. S. MEYERS,[16] AND E. A. SCHULTZ,[17] ASSOC. MEMBERS, ASCE

SYNOPSIS

A current of about 4.5 knots, at the Atlantic end of the Panama Canal, is estimated both by computations and by hydraulic model tests as the maximum that would be caused in an open sea-level canal at Panama by a tidal range of 20 ft in the Pacific Ocean.

Channel roughness would affect the velocity of the reversing flow in this tidal canal in the same way as in any one-way channel since, at the time of maximum current, the situation is nearly that of ordinary steady flow between the extreme tide levels. Results of measurement of roughness in the existing canal and a summary of roughness data for other large channels are presented. A Manning n of 0.024 was selected for the investigations.

The hydraulic model of the sea-level canal, at 1:100 undistorted scale, is described briefly. Close agreement throughout the tidal cycle was obtained between the velocities measured in the model and those computed by the method developed by General Pillsbury.

An example of tidal flow as controlled by tidal-regulating structures is presented and discussed briefly.

EARLY ESTIMATES OF TIDAL CURRENTS AT PANAMA

The currents that would be produced by the tides in a sea-level canal at Panama have been the subject of speculation and concern since the earliest canal proposals. The first careful study known to have been made was that reported on May 31, 1887, by a committee appointed by the French Academy of Sciences[18,19] at the request of Count Ferdinand de Lesseps, chief engineer of the French Isthmian Canal Company, who was undertaking to build a sea-level canal at Panama as he had already done so successfully at Suez. The French analysis was based on the use of the Chézy formula for steady flow, applied to a number of short reaches, with heads adjusted to take account of wave celerity. The committee reached the conclusions that the Atlantic tidal range was so small in comparison with the Pacific range that it could be disregarded. The estimated maximum tidal current for a canal 45 miles long was 2.5 knots.[20]

The Board of Consulting Engineers, Isthmian Canal Commission, reported in 1906: "It is probable that in the absence of a tidal lock the tidal currents during extreme oscillations would reach five miles per hour."[21] This is the

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[18] "The Cape Cod Canal," by William Barclay Parsons, *Transactions*, ASCE, Vol. LXXXII, 1918, p. 138.

[19] *Comptes Rendus*, French Academy of Sciences, Vol. 104, May 31, 1887, p. 1484.

[20] *Transactions*, ASCE, Vol. LVI, 1906, p. 211.

[21] "Report of the Board of Consulting Engineers for the Panama Canal, 1906," U. S. Govt. Printing Office, Washington, D. C., 1906, p. 56.

equivalent of 4.3 knots. The channel to which this estimate applied was about 40 miles long and 40 ft deep, with bottom width at 150 ft in earth and 200 ft in rock. The method used to obtain this velocity was not stated.

An estimate of 2.6 knots for the mean cross-sectional velocity in a sea-level canal was made by the United States Coast and Geodetic Survey (in a letter to the Panama Canal, dated February 3, 1924), for a channel 40 miles long, 1,000 ft wide, and 50 ft deep. The Eytelwein formula was used to compute the steady-flow velocity that would be produced by an 11-ft difference in head. It was stated that the velocities computed by this formula agreed reasonably well with observed values in the Cape Cod Canal in Massachusetts, which is about 7 miles long, but that progressive wave motions might be created in the greater length of a Panama sea-level canal which would tend to increase the computed value.

TIDES AT PANAMA

The tides at the Atlantic and Pacific termini of the Panama Canal are very different both in magnitude and general character. The Pacific tide at Balboa is remarkably regular, with two highs and two lows of almost equal magnitude occurring in every lunar day of 24 hours and 50 min. Extreme tides have reached levels 10.8 ft above and 11.9 ft below mean sea level. The maximum range between consecutive tides, however, has only occasionally exceeded 20 ft, as indicated by the following percentages of Balboa tides that reach different ranges:

Percentage of total tides	Tidal range (ft)
2	20
20	16
50	13
80	10
99	6

The Atlantic tide at Cristobal is irregular and much smaller than the Pacific tide, with no simple cyclic pattern. Extreme tides have reached 1.8 ft above and 1.25 ft below mean sea level. The mean tidal range is 0.9 ft and the minimum practically zero. Atlantic high tides precede Pacific high tides from zero hours to 6 hours, averaging about 3 hours. Mean sea level on the Pacific side of the Isthmus averages 0.77 ft higher than on the Atlantic side, but the Pacific mean level on individual days has ranged from 2.04 ft above to 0.68 ft below the Atlantic mean level.

Most of the analytical and model studies described in this paper were based on assumed maximum semidiurnal tides of 20-ft range at Balboa and of 2-ft range at Cristobal, with the Cristobal tide preceding the Balboa tide by 3 lunar hours, as shown in Fig. 26. (One lunar day is the time required for one revolution of the moon around the earth. It corresponds with 24 hours 50 min of ordinary solar time.) Selection of the essential features of a sea-level canal generally required consideration of the conditions produced by these maximum

tides, but investigations were also made of the effects of average and low tides for some comparisons.

Because the Pacific mean sea level is 0.77 ft higher than the mean level on the Atlantic side of the Isthmus, a greater head and a steeper slope in a sea-level canal would normally be expected when the Pacific tide was at its highest point than when it was lowest. The maximum head in 20 years of record,

however, was 11.6 ft from the Atlantic level down to a low Pacific tide level. The maximum head in the opposite direction, from a high Pacific level down to the Atlantic level was 11.3 ft. Since the tidal heads in the two directions are so nearly the same for maximum conditions, the convenient assumption of a common Atlantic and Pacific mean tide level was made for study purposes.

ANALYSES OF TIDAL CURRENTS FOR 1947 INVESTIGATION

Estimates of velocity for the proposed 60-ft by 600-ft sea-level canal have been made,

FIG. 26.—MAXIMUM TIDES USED FOR VELOCITY STUDIES

using data and methods more complete than those available to earlier investigators. Most of the computations were made by the method developed by General Pillsbury for analysis of tidal flow.[8] By this method the channel is subdivided into a number of reaches, and flow in each reach is computed from an equation of motion in which the frictional resistance is expressed by the Manning, the Kutter, the Bazin, or any desired flow formula. Corrections are made for changes in storage, slopes, and other factors caused by the changing tides. The precision of the results depends on the number of successive adjustments. This method indicates that a maximum current velocity of 4.5 knots would be caused at the Atlantic end of the canal by the tides of Fig. 26 acting on an open sea-level canal with the Manning roughness assumed as 0.024. This velocity, and all others given in this paper, whether from model tests or computations, are mean velocities over the entire channel cross section.

A computation procedure suggested by Professor Bakhmeteff was also used for comparison. The basic assumption was that of steady flow between the maximum differences of water level shown in Fig. 26, with corrections for changes in acceleration, velocity head, and channel storages that would prevail in a tidal cycle. The estimated maximum steady-flow velocity was 4.6 knots, again using a Manning roughness of 0.024.

Tests on the 1:100 scale hydraulic model of the proposed sea-level canal channel, which is described subsequently in this paper, indicated a maximum
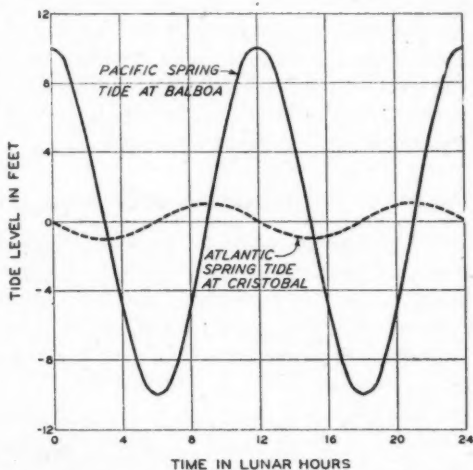
current of 4.4 knots, for the same 20-ft and 2-ft tides. The roughness of the model channel corresponded to that expressed by a 0.024 Manning coefficient for the full-size channel.

During the preliminary discussions of these analyses and their implications, Professor Bakhmeteff stated that velocities for tidal flow cannot exceed those for steady flow. For the situation at Panama, he reasoned that the steady-flow velocities would be only slightly greater than the tidal velocities, since "preliminary calculations show that at periods of extreme flow about 90 per-cent of the actuating head is absorbed by friction resistance." The results of the tests on the sea-level model closely support that statement, indicating that channel roughness is a controlling factor in any estimate of tidal velocities in a canal as long as that at Panama.

HYDRAULIC ROUGHNESS OF LARGE CHANNELS

Observations made on open channels of moderate and small size have provided most of the data upon which the current understanding of flow is based. The flow formulas in common use were developed primarily from mea-surements made on small test channels by Bazin and other early experi-menters. In some cases, they were developed with a view to their use for

TABLE 19.—VALUES OF MANNING'S ROUGHNESS COEFFICIENT FOR LARGE CHANNELS AT MOST EFFICIENT DEPTHS

Line	Watercourse	Coefficient n	Notes on channel
1	Emory River...............	0.029	Earth and ledge rock.[a]
2	Tennessee River............	0.026	Natural earth channel of lower Tennessee River.[b]
3	Colorado River.............	0.028	In newly excavated rock.[c]
4	Mississippi River...........	0.024 to 0.031	In alluvial valley.[d]
5	Atchafalaya River..........	0.024	In alluvial valley.[e]
6	Yazoo River................	0.026 to 0.031	In alluvial valley.[f]
	Columbia River—		
7	McNary dam site near Uma- tilla, Ore..............	0.030	Stable bottom of sand, gravel, boulders, and solid rock.
8	Central part of State of Washington	0.036	Little bank vegetation.
9	Cape Cod Canal............	0.031 to 0.036	Subject to tidal currents.
10	Houston Ship Canal........	0.025	Coefficient measured roughly.[g]
11	Panama Canal (Gaillard Cut)	0.026	In rock.[h]

[a] In backwater from Watts Bar reservoir on the Tennessee River below Knoxville, Tenn.   [b] In backwater from Kentucky Dam near Gilbertsville, Ky.   [c] Downstream from powerhouse at Parker Dam near Parker, Ariz.   [d] This range of values was applicable to 1928–1930 measurements, omitting widely divergent values.   [e] This range of values was applicable to 1929, omitting widely divergent values.   [f] After clearing and snagging. Climate is conducive to vegetation.   [g] This coefficient, although roughly measured, is considered applicable to design.   [h] Observations in 1946, after channel had been submerged for 33 years.

closed pipes as well as open channels. Although the desirability of having the formula fit data for large as well as for small channels was recognized, notably by Ganguillet and Kutter, the information then available on large channels was meager and in some cases unreliable. Because of the importance of the roughness value in this Panama Canal study, special effort was made to obtain and compare information on large channels only, and to weigh the applicability of the different formulas to those conditions.

Roughness Data on Large Channels from Other Sources.—In response to requests made in 1946 by The Panama Canal to a number of agencies in the United States, information on hydraulic roughness of large channels was received from the Bureau of Reclamation, the Tennessee Valley Authority, and several offices of the Corps of Engineers. The data were for long reaches of natural and artificial channels with a considerable variety of slopes, discharges, depths, and bed material. The roughness coefficients have been averaged, and a summary for channels having hydraulic radii exceeding 20 ft and flowing nearly bankfull is given in Table 19.
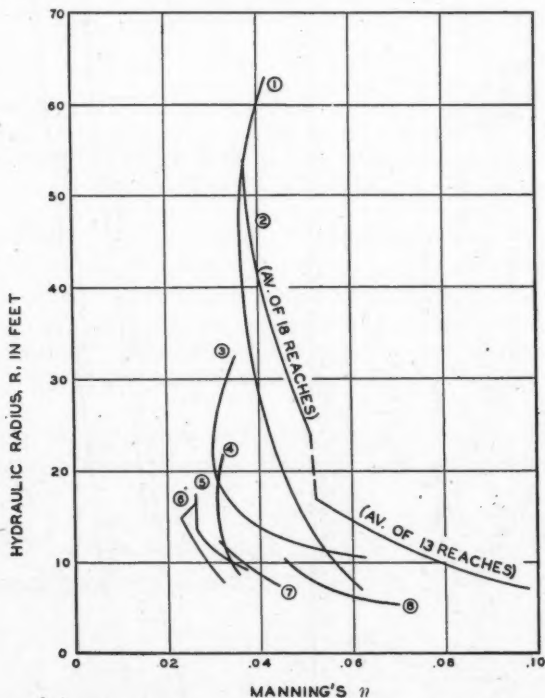
Fig. 27.—Variation of Manning's n, with Hydraulic Radius

The two most important conclusions reached from a study of these data were:

a. The Manning coefficient for a river channel is least when the stage is at or somewhat above normal bankfull stage, and tends to increase for both higher and lower stages. This fact is indicated both by the Yazoo River in Mississippi, flowing in alluvium in an area where vegetation is rank, and by the Columbia River in Washington, with a rocky bed and practically no vegetation on its banks. The effect is shown in Fig. 27, the several curves being identified by numbers as follows:
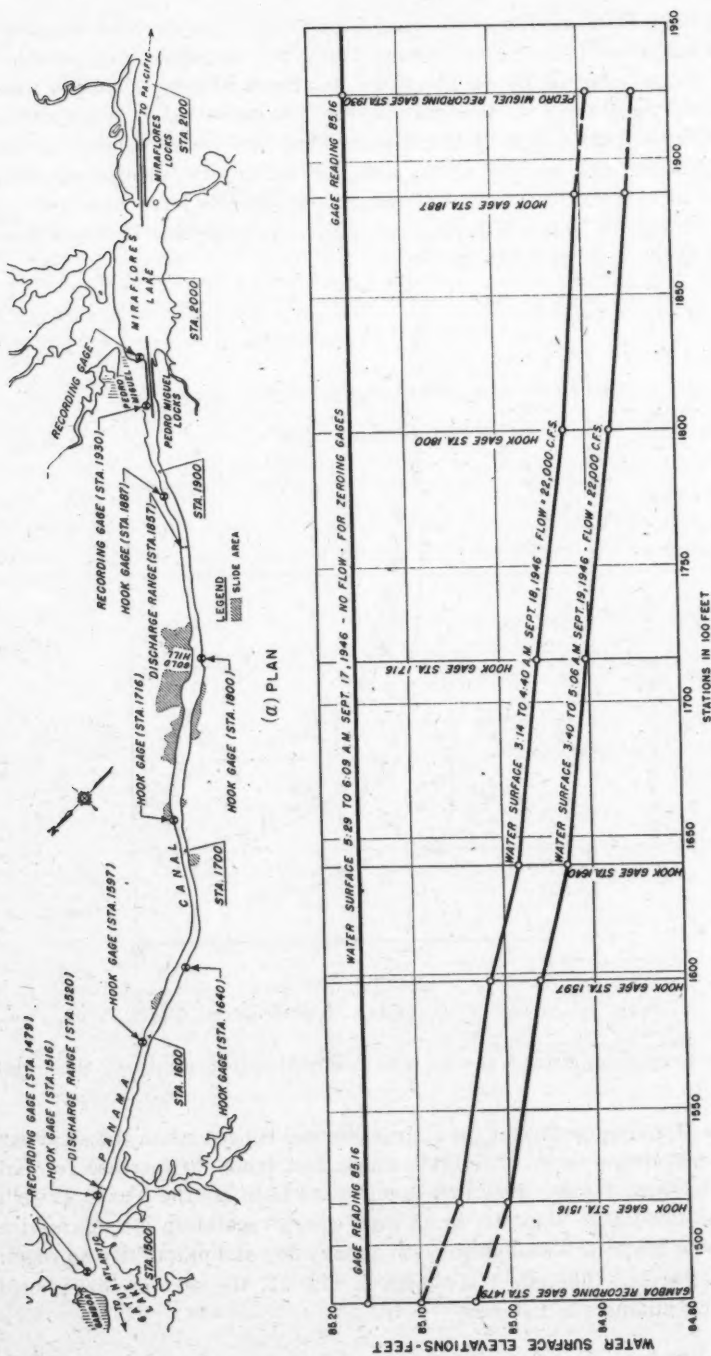
(a) PLAN

(b) WATER SURFACE PROFILES

Fig. 28.—Roughness Measurements in Gaillard Cut

Curve	Identity
	Columbia River—
1............	Foster Creek project above Wenatchee, Wash.
2............	Priest Rapids project near Ellensburg, Wash.
3............	McNary project above Umatilla, Ore.
	Yazoo River—
4............	At Yazoo City, Miss.
5............	At Greenwood, Miss.
	Tallahatchie River—
6............	At Swan Lake, Miss.
7............	At Lambert, Miss.
	Big Black River—
8............	At West, Miss.

b. The bankfull roughness coefficients do not vary greatly for rivers and canals in different kinds of material and in widely separated locations. From the summary in Table 19 for channels having hydraulic radii exceeding 20 ft, all but one show roughness coefficient values in the range from 0.024 to 0.031.

Roughness Measurements in Gaillard Cut.—Measurements were made of the roughness of the Gaillard Cut channel of the existing canal, to obtain data under conditions as nearly comparable as possible to those in a new canal channel. A flow of water through the cut was established by opening all the culvert valves at Pedro Miguel Locks after the last ship transit in the evening, and continued until necessary to shut down before the first transit in the morning.

TABLE 20.—ROUGHNESS COEFFICIENTS, GAILLARD CUT

| Reach (stations)	Difference in Water-Surface Elevation		Average energy gradient slope, S	Mean section area (sq ft)	Mean velocity (ft per sec)	Mean hydraulic radius, R (ft)	Chézy C	Manning n (a)	Kutter n	Modified Kutter n	Bazin m	Kármán-Prandtl k
	Sept. 18	Sept. 19										
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
1516–1597	0.040	0.041	0.0000060	14,900	1.48	36.5	101	0.0268	0.052	0.025	3.4	0.35
1597–1640	0.033	0.032	0.0000052	15,800	1.39	36.4	100	0.0271	0.053	0.024	3.5	0.38
1640–1716	0.025	0.025	0.0000055	16,500	1.34	37.4	101	0.0270	0.053	0.024	3.4	0.37
1716–1800	0.033	0.031	0.0000023	21,500	1.02	38.0	108	0.0252	0.054	0.022	2.8	0.22
1800–1887	0.021	0.024	0.0000046	18,000	1.22	36.0	103	0.0261	0.052	0.023	3.1	0.29
1516–1640	0.073	0.073	0.0000057	15,200	1.44	36.4	101	0.0269	0.053	0.024	3.4	0.36
1640–1887	0.079	0.080	0.0000041	18,600	1.18	37.2	103	0.0263	0.053	0.023	3.2	0.31
1516–1887	0.152	0.153	0.0000046	17,200	1.28	37.0	102	0.0265	0.053	0.023	3.3	0.33

ᵃ From water-surface profile computations. The remaining coefficients have been determined from n, R, and S as tabulated. The slope term, $0.00281/S$, of the Kutter equation is omitted in the modified Kutter equation.

Six temporary gages were established for these tests and their zero elevations were determined by simultaneous readings of all gages during a preliminary run with quiet water surface and no flow through the cut. An average dis-

charge of 22,000 cu ft per sec was measured by current meter at each end of the test reach for each of the two tests.

Areas and hydraulic radii for the channel cross sections were obtained from a detailed survey made in 1945. Fig. 28 shows the locations of gages and the observed water-surface profiles, and Table 20 lists the coefficient values found for different flow formulas. The depth of the channel was quite uniformly 41 ft and the minimum bottom width was 300 ft, but the width and cross-sectional area varied considerably in reaches where the channel passes through bends and old slide areas. The nature of the bed and bank material varies from very hard rock to soft rock and clay. Fig. 29 shows the nature of the channel just before water was admitted, superimposed lines indicating the ultimate water level.

FIG. 29.—CULEBRA CUT (LATER NAMED GAILLARD CUT) AT EMPIRE,
AFTER THE COMPLETION OF EXCAVATION

The measurements were made with great care, working at night to avoid interference from traffic and wind, and allowing several hours on each of the two test dates for water levels to become well stabilized. ●Roughness coefficients for each of the five reaches into which the 7-mile test section was subdivided are very consistent, as shown by Table 20. Observed coefficient values of 0.0265 for the Manning formula, 3.3 for the Bazin formula, and 0.33 for the Kármán-Prandtl formula fall within the expected range. The unmodified Kutter formula, however, gave the high coefficient value of 0.053 because of the very flat slope of 2 in. in 7 miles. An n-value of 0.023 was indicated for the Kutter formula with the slope term omitted.

Selected Value for Roughness Coefficient.—The sea-level canal channel would have a mean hydraulic radius of about 55 ft, and would be cut through a variety of materials ranging from muck to basalt, but at least 87% of the length would fall within the classifications of hard, medium, and soft rock. Therefore, a roughness coefficient representative of a rock surface like that of the existing Gaillard Cut should be used.

The Manning formula was adopted for use in the studies for the improvements of the Panama Canal. The range of Manning roughness coefficients
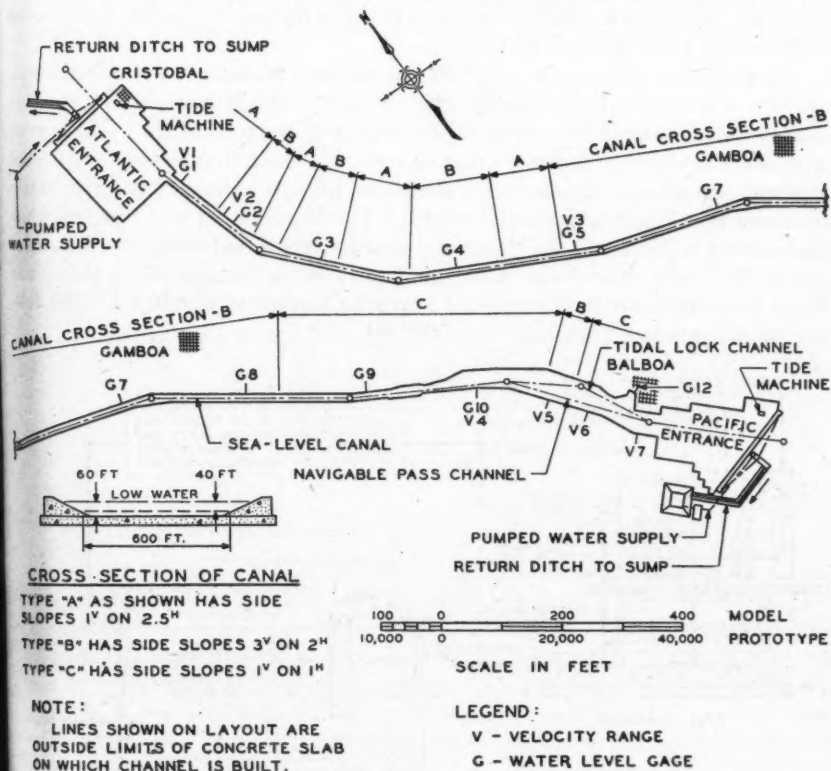
FIG. 30—LAYOUT OF 1:100 MODEL OF PANAMA SEA-LEVEL CANAL

from all sources is generally between 0.024 and 0.031, which include the Gaillard Cut observations of 0.026. A coefficient of 0.024 was selected for the computation of velocities in a sea-level canal, where the use of the highest velocity that could reasonably be expected would result in a more conservative design.

MODEL OF SEA-LEVEL CANAL

Fig. 30 shows the layout for the hydraulic model channel that was built and tested in the Canal Zone for the study of maximum tidal currents and of the general hydraulic problems that would arise in the design and operation of a

sea-level canal. The entire length of the proposed 60-ft by 600-ft navigation channel was reproduced, and also the Atlantic and Pacific entrances out to deep water, to insure accurate simulation of flow into and out of the canal. At the scale of 1:100, the model was nearly half a mile long, simulating a total prototype length of about 45 miles and including some 35 miles of restricted channel. Varying side slopes were used, to agree with the design slopes for the different geological materials through which the channel would be cut. The model was built outdoors, with a cover of corrugated sheet metal laid across the top of the channel side walls and roofs over the entrances to prevent disturbance of the water levels by rain and wind.

Model Scale.—The scale of 1:100 was selected because it permitted satisfactory measurement of velocity and of differences in water level and because it was expected to reproduce the flow conditions correctly. Experience with other models had indicated that an ordinary concrete surface in a model of this scale would simulate the roughness of the full-size channel more accurately than would a larger or a smaller model. This expectation was fulfilled when the resistance coefficient for the model channel, measured under conditions of steady flow with differential head equal to that for the maximum tides, was found to correspond to the selected Manning coefficient of 0.024 for the full-size canal, and no adjustments were needed.
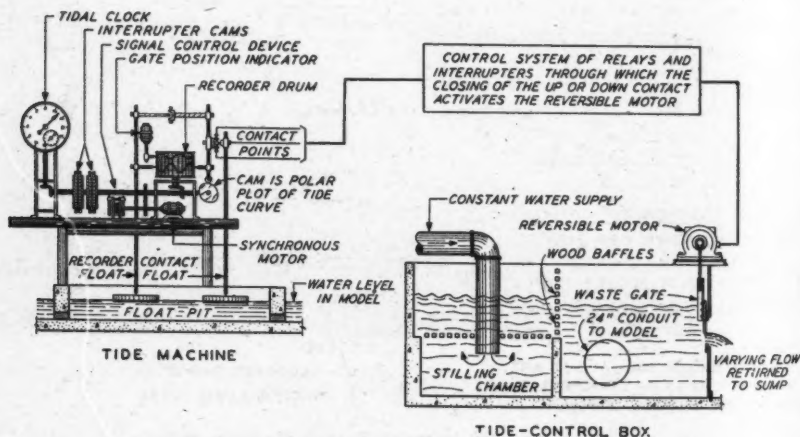
FIG. 31—SCHEMATIC LAYOUT OF TIDE-REPRODUCING APPARATUS, MODEL OF SEA-LEVEL CANAL

The model was built without distortion to insure, as far as possible, that dynamic similitude was obtained, because the absence of an existing prototype prevented making the extensive tests that are essential for the verification of a distorted model. Since gravity is the predominant force in this case, the time, discharge, and velocity factors were used strictly in accordance with the Froude law, which considers gravity as the only factor affecting the motion of the water. Flow at the Atlantic end of the model for maximum tidal conditions would meet the Reynolds' criterion for turbulence about 96% of the time

(based on a Reynolds number of 2,000).[22]   This situation covers all the critical conditions where accuracy is needed.   Regardless of scale, laminar flow cannot be avoided entirely in this model since the velocity must be zero at each reversal of the tide.

Tide Machines.—Tide-control mechanisms were installed at each end of the model to reproduce the tidal variations of the prototype to proper time and height scales.   The machines were made by the United States Waterways Experiment Station at Vicksburg, Miss., following the design which had proved satisfactory in other models, and installed at Panama under their supervision of Experiment Station personnel.   Figs. 31 and 32 show the tide-
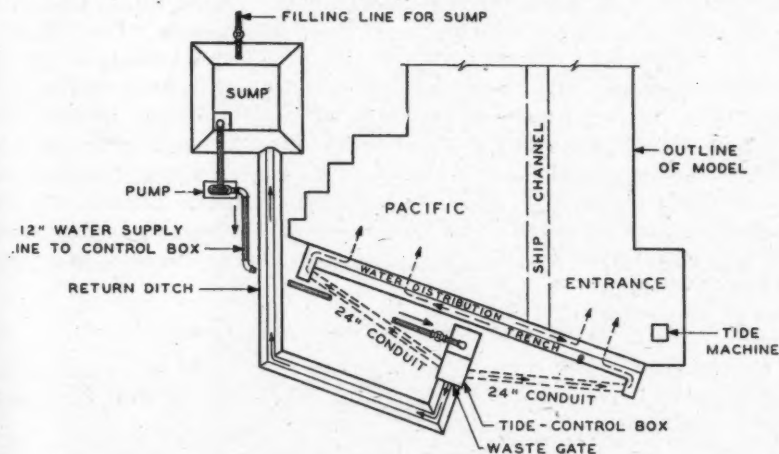
FIG. 32.—WATER-SUPPLY SYSTEM AT PACIFIC END, MODEL OF SEA-LEVEL CANAL

reproducing apparatus and the water-supply system at the Pacific entrance. The solid arrows in Fig. 32 indicate continuous flow in one direction, whereas dashed arrows indicate reversing flow that changes in direction with every change in tide.   The basic element of each tide machine is a replaceable cam cut to represent the desired tidal cycle.   Mechanical and electronic devices operate a waste gate in the water-supply system which controls the rate of model inflow or outflow so that the actual water surface closely follows the indications given by the cam.

Principle of Tidal-Model Operation.—The principle of operation for this model was to control the water levels at the two ends to follow the desired tides, and to make observations of the water flowing back and forth in the channel under the influence of those tides.   Any desired combination of tides could be used, and any desired modifications could be made in the operation of tidal-regulating structures.   To observe the effects of any modification, it was always necessary to operate the model through one or more complete tidal cycles, and to coordinate a series of measurements at frequent intervals at a number of locations.   This procedure differs essentially from that used on the usual river model, where tests are normally made with steady flows, and

[22]"Hydraulic Models," *Manual of Engineering Practice No. 25*, ASCE, 1942, p. 36.

leisurely adjustments and observations can be made until the experimenter is certain that a test run is satisfactory. To repeat any measurement on a tidal model, however, a full tidal cycle must first be run—which, for a 1:100 scale, requires 1.25 hours—and a crew of observers must then make the desired measurements in as short a time as possible, before conditions change appreciably.

Model Measurements.—Fig. 30 shows the locations at which observations were made in the model. Water levels at the extreme ends were measured by recording gages which form part of each tide machine, and by automatic recorders and manually operated hook gages along the canal channel. Velocities were measured with pigmy current meters of the United States Geological Survey type, which were recalibrated at frequent intervals. For velocities lower than about 0.2 ft per sec (corresponding to full-scale velocity of 2 ft per sec), the meters were supplemented by measurements with floats consisting of corks supporting metal vanes suspended at a six-tenths depth. Observations were made simultaneously by observers at the different stations, at intervals varying from 5 min to 30 min of prototype time—the frequency of the observation depending on its importance and rapidity of change. Electric bells and signal lights were actuated by the tide machines at intervals corresponding to 30 min of prototype time—about 3 min of actual time—to assist in correlating the observations.

CURRENTS IN AN OPEN SEA–LEVEL CANAL

As previously stated, the maximum current that would be created at the Atlantic end of an open sea-level canal by the combination of 20-ft Pacific and

ATLANTIC END   PACIFIC END

MEAN TIDE LEVELS
DEPTH=60 FT   WATER SURFACE   10 FEET
              V_{MAX} = 3.7 KNOTS   DEPTH=60 FT
              Q_{MAX} = 240,000 C.F.S. V_{MAX} = 3.7 KNOTS
                                               Q_{MAX} = 220,000 C.F.S.
                          CANAL BOTTOM

(a) FLOW TOWARD PACIFIC WITH
     LOW PACIFIC TIDE

NOTE:
  VELOCITIES AND FLOWS ARE FROM
  TESTS ON SEA-LEVEL MODEL
                          WATER SURFACE   10 FEET
                          MEAN TIDE LEVELS
DEPTH=60 FT   DEPTH=80 FT
     V_{MAX} = 4.4 KNOTS
     Q_{MAX} = 290,000 C.F.S. V_{MAX} = 3.3 KNOTS
                                       Q_{MAX} = 270,000 C.F.S.
                          CANAL BOTTOM

(b) FLOW TOWARD ATLANTIC WITH
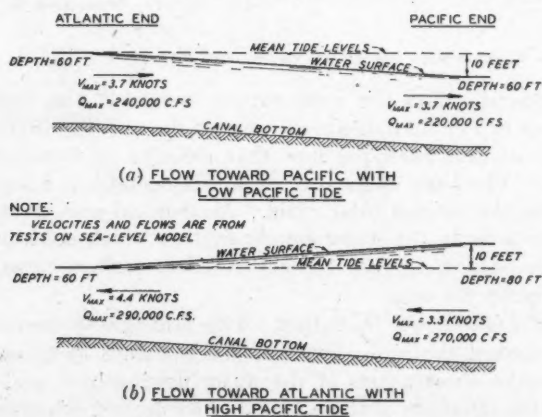   HIGH PACIFIC TIDE

FIG. 33.—LOCATION OF MAXIMUM VELOCITIES FOR TIDAL FLOW

2-ft Atlantic tides has been indicated as 4.4 knots by the sea-level model, as 4.5 knots by the Pillsbury method of computation, and as 4.6 knots by steady-flow computations.

Location of Maximum Velocity.—Fig. 33, showing velocities and discharges taken from a test on the sea-level model, indicates how the maximum velocity

occurs at the opposite end of the canal from the strong tide that creates it. The upper diagram indicates conditions for maximum flow toward the Pacific, when depths and velocities are nearly constant throughout the canal. With maximum flow toward the Atlantic, however, as indicated in Fig. 33(b), the depths and velocities vary considerably, the discharge is greater, and the highest velocity occurs at the point of smallest cross-sectional area, which is at the Atlantic end of the channel.

Fig. 34 shows water-surface profiles as observed in the model for successive hours during the tidal cycle. The changing slopes indicate the direction and relative velocity of the flow. When the Pacific tide was high (hour 12, Fig. 34), water throughout the length of the canal would flow toward the Atlantic.
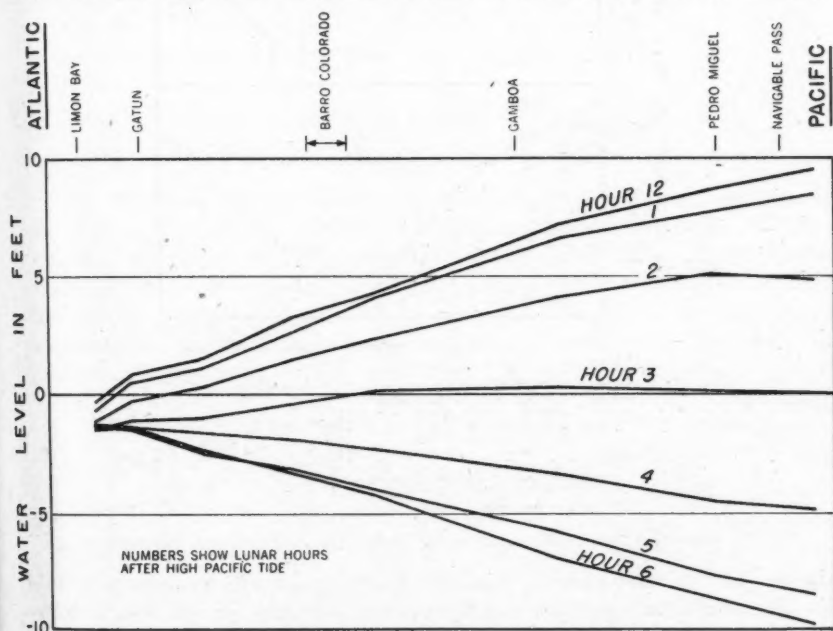
FIG. 34.—WATER-SURFACE PROFILES FOR AN UNREGULATED 20-FT PACIFIC TIDE, MODEL OF AN OPEN SEA-LEVEL CANAL

As the Pacific tide dropped, the flow would slacken and then change direction. For a short period during the change, water would flow out of each end of the canal, and then the flow throughout the canal would gradually increase to maximum strength toward the Pacific (hour 6, Fig. 34). A similar sequence of flows in the opposite direction would occur during the next half-cycle of 6 hours while the tide returned to its starting point. The current in the canal would always vary gradually and continuously with no abrupt differences, and would always change direction every 6 hours. A sinusoidal curve would be a good representation of the velocity at any point.

Comparison of Model Velocities and Computed Velocities.—Fig. 35 shows the observed velocities for unregulated tidal flow at the measuring stations nearest

the ends of the model, and also the velocities as computed by the Pillsbury method when carried to different degrees of refinement. The agreement is very close throughout the tidal cycle as well as at the maximum points. The "second adjustment" computations which show the best agreement were carried out only for the combination of 20-ft and 2-ft tides. For most of the analyses

LEGEND:

	COMPUTED VELOCITY, PRIMARY CURRENT, FIRST RECOMPUTATION
	COMPUTED VELOCITY, FIRST ADJUSTMENT
	COMPUTED VELOCITY, SECOND ADJUSTMENT
o	MEAN VELOCITY MEASURED IN SEA-LEVEL MODEL, TESTS OF JUNE 20 AND 27, 1947

NOTES:

1. CONDITIONS: 2-FOOT ATLANTIC RANGE PRECEDING A 20-FOOT PACIFIC RANGE BY 3 HOURS.

2. COMPUTATIONS ARE BY PILLSBURY'S METHOD
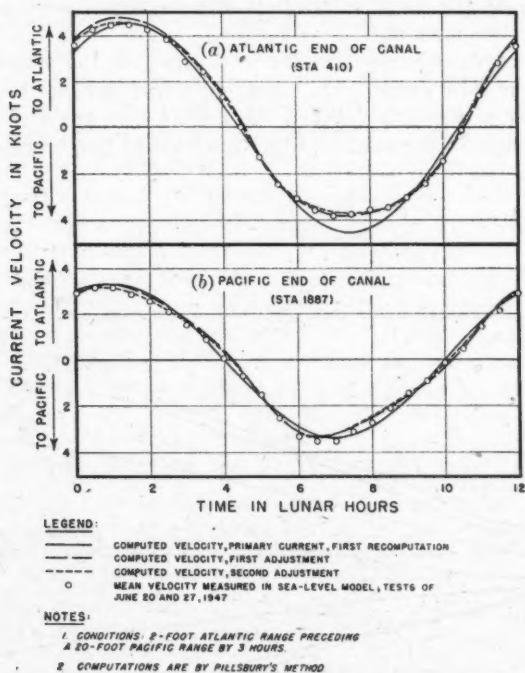
FIG. 35.—COMPUTED AND OBSERVED CURRENT VELOCITIES, UNREGULATED SEA-LEVEL CANAL

the simpler "primary current" computation was used, which agrees very closely for the maximum velocities and is approximate only for the less important smaller velocities. In applying his computation method to an example of tidal flow in a Panama sea-level canal, General Pillsbury (in a letter to The Panama Canal, dated February 1, 1946) has concluded:

(1) " * * * that the corrections produced from the somewhat extensive computations required for the second adjustment are quite small and are in fact well within the uncertainties inherent in the selection of the proper coefficient of roughness, and in the dimensions of the canal as actually dredged, and as eroded by the strong currents";

and that

(2) " * * * primary currents afford a reliable indication of the magnitude of the currents to be expected in an open sea-level canal across the Isthmus."

Table 21 shows a summary of computed (the Pillsbury primary current) and model velocities for the open sea-level canal. All velocities are the maxima

reached during the tidal cycle. The model values listed are the average of velocities measured at seven points across the width of the channel. Velocities at the center of the canal (measured at six-tenths depth) are about 5% greater than these averages.

TABLE 21.—COMPARISON OF COMPUTED AND MODEL VELOCITIES FOR MAXIMUM TIDAL CURRENTS IN UNREGULATED SEA–LEVEL CANAL

TIDAL RANGE (FT)		Computed velocity at Atlantic end (knots)	OBSERVED VELOCITY IN MODEL (KNOTS)	
Pacific	Atlantic		Atlantic end	Pacific end
20................	2	4.5	4.4	3.7
16................	1	4.0	3.8	3.0
13................	1	3.5	3.3	2.5
10................	1	3.0	2.7	2.0
6................	1	2.1	2.1	1.5

Fig. 36 shows, for any time during a tidal cycle and for any position along the channel, the velocity that would be found at that point. The "contours," which were drawn beween values observed for several cycles in the sea-level model, indicate the regimen of velocity for the combination of 20-ft Pacific tides and 2-ft Atlantic tides. Sets of diagrams of this kind, for the full range of tides, have been useful in navigation studies, since lines can be drawn upon them to represent ship transits at different speeds and directions.
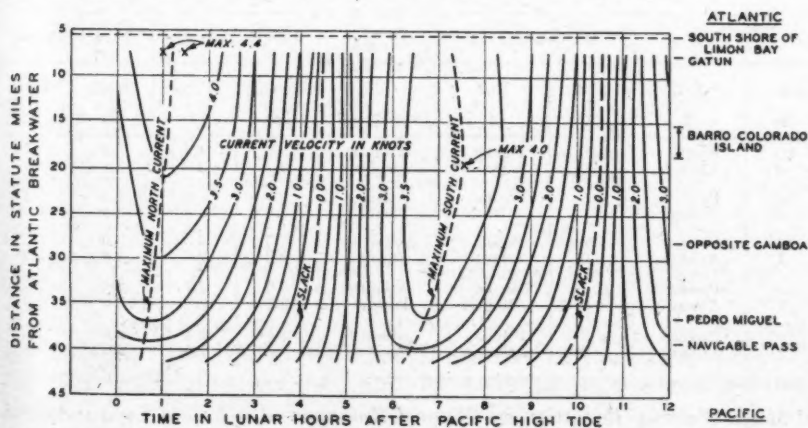
FIG. 36.—TIME-POSITION-VELOCITY DIAGRAM; UNREGULATED FLOW, 20-FT PACIFIC TIDE

New Flow Toward the Atlantic.—An indicated in Fig. 33, there would be a greater flow toward the Atlantic during half of each tidal cycle than toward the Pacific during the other half. The water in the channel would move alternately toward the Atlantic and then toward the Pacific with the changing tides, with a net advance of about 5 miles per day toward the Atlantic indicated by

the model for the 20-ft and 2-ft tides.   For the same tidal ranges, but with the Pacific mean level higher than the Atlantic mean level, the total movement would increase.   The most extreme situation of this kind which was found was that of December 19, 1937, when the 19.44-ft Pacific tide range averaged 1.74 ft higher than the Atlantic mean level, and the computed net movement of water (a second adjustment computation made by General Pillsbury) would have been about 20 miles per day.

Comparison of Steady-Flow and Tidal-Flow Velocities.—Table 22 compares maximum velocities for steady flow and tidal flow for different tidal ranges, as observed in the sea-level model.   With minor exceptions (which are attributable to experimental error), steady-flow velocities are slightly higher. Computed velocities for steady flow agree closely with model results.   The data support the opinion that tidal velocities should not exceed velocities for steady flow between the corresponding extreme water levels.   Velocity computations on the assumption of steady flows are not difficult to make, but this procedure indicates only the maximum velocity in either direction.   The Pillsbury method, on the other hand, provides successive values for velocities and water levels at all times during the tidal cycle.   For this reason it was generally used as the basic computation method for analytical studies of velocities in the sea-level canal.

TABLE 22.—Tidal-Flow Velocities and Steady-Flow Velocities in Sea-Level Model

WATER LEVELS				End at which velocities are measured	MAXIMUM VELOCITY (KNOTS) FOR:			
For Tides		For Steady Flow			Flow to Atlantic		Flow to Pacific	
Feet	End	Feet	End		Steady	Tidal	Steady	Tidal
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
20	Pacific	±10	Pacific	Atlantic	4.6	4.4	3.5	3.7
2	Atlantic	0	Atlantic	Pacific	3.7	3.3	3.8	3.7
16	Pacific	± 8	Pacific	Atlantic	3.7	3.8	3.3	3.4
1	Atlantic	0	Atlantic	Pacific	3.3	2.8	3.4	3.0
13	Pacific	± 6.5	Pacific	Atlantic	3.3	3.3	3.0	3.1
1	Atlantic	0	Atlantic	Pacific	2.9	2.4	3.0	2.5
10	Pacific	± 5	Pacific	Atlantic	3.0	2.7	2.6	2.5
1	Atlantic	0	Atlantic	Pacific	2.4	2.0	2.6	2.0
6	Pacific	± 3	Pacific	Atlantic	2.4	2.1	2.0	1.8
1	Atlantic	0	Atlantic	Pacific	1.7	1.5	2.0	1.4

Minor Velocity Reduction by Channel Enlargement.—As has been indicated in Fig. 33, the maximum tidal current in a sea-level canal with constant width and constant depth below a line connecting the low water levels would occur at the Atlantic end where the cross-sectional area is least.   This velocity could be decreased to some extent by deepening or widening the canal at the Atlantic end.   Any such enlargement, however, would increase the total flow capacity of the channel, and the velocities would be increased throughout the tidal cycle in the part of the channel not enlarged.

The greatest reduction in velocity could be obtained by a gradually tapered enlargement for about two thirds of the length of the canal, starting near Gamboa and extending to the Atlantic end, where the cross section would need to be enlarged by 16%. The maximum current for a 20-ft Pacific tide could be reduced from 4.5 knots (as computed by the Pillsbury method) to 4.0 knots which would then prevail for most of the length of the canal—the current at the Pacific end being increased while that at the Atlantic end was decreased.

CURRENTS IN A REGULATED SEA–LEVEL CANAL

Reduction of Current by Tidal-Regulating Structures.—Tidal currents in a sea-level canal could be regulated effectively by the structures shown in Fig. 9. The maximum current could then be regulated to any desired limiting value between 0.5 knots and 4.5 knots. The value of 0.5 knot would be the current produced by the action of a 2-ft Atlantic tide in a canal closed at the Pacific end, and the value of 4.5 knots would be that produced by the combination of 20-ft Pacific and 2-ft Atlantic tides in the open channel. Fig. 37 shows the
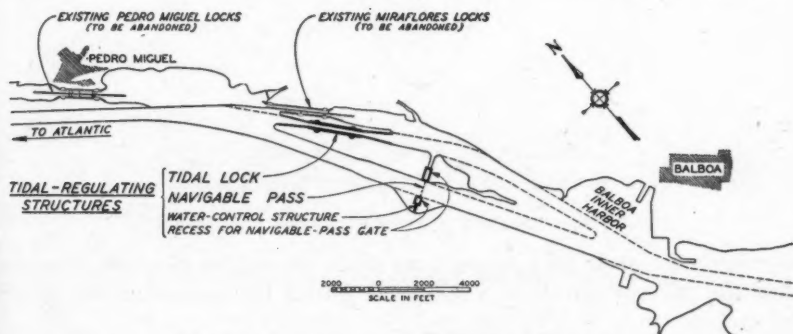
FIG. 37.—LAYOUT OF TIDAL-REGULATING STRUCTURES

proposed layout for a tidal lock and navigable pass. The pass would normally be closed when the tide was at a high level or a low level to exclude the entry of tidal flow into the canal which would cause currents in excess of the selected maximum value. Ships would use the tidal lock when the navigable pass was closed.

Fig. 38 shows schematically the operation of the regulating structures during a tidal cycle. The action of such structures may be considered as replacing the natural Pacific tide (range A-D, Fig. 38) with a smaller regulated tide (range B-C) on the canal side of the structures which would produce currents not exceeding the selected limits. For a 2-knot limit the regulated tide would be about 6 ft. For a higher limiting current, the regulated range would be greater and the navigable pass could be kept open longer. The water level in the canal would return approximately to Atlantic tide level if no water were released through the tidal-regulating structures after point C, Fig. 38. Reopening of the navigable pass would then be delayed beyond point E until the natural tide level rose to meet the canal level. The function of the water-control

structure is to permit the release of a limited flow of water from the canal to the Pacific during this period and to bring the canal level to the desired elevation at point E. The navigable pass could then be opened promptly on schedule,
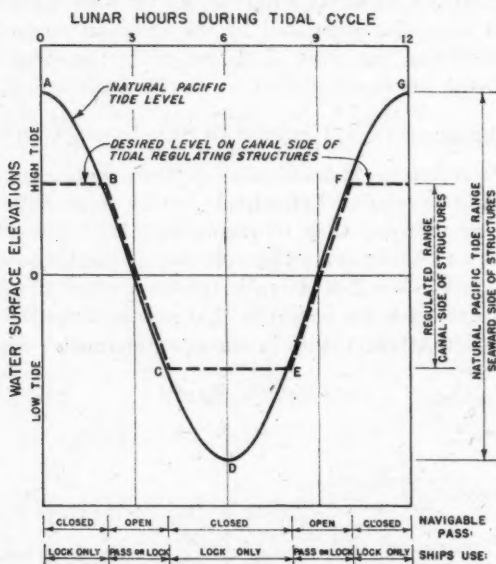

FIG. 38.—OPERATION OF TIDAL-REGULATING STRUCTURES

and would stay open for a longer total time. A similar sequence of events, but in the opposite direction, would occur during the remaining half of each tidal cycle.

On the basis of the idealized conditions indicated in Fig. 38, the permissible ranges of water levels on the canal side of the tidal-regulating structures were

TABLE 23.—DAILY AVAILABILITY OF NAVIGABLE PASS

Permissible current (knots)	HOURS PER DAY NAVIGABLE PASS COULD REMAIN OPEN FOR PACIFIC TIDAL RANGES OF:				
	6 ft	10 ft	13 ft	16 ft	20 ft
1.............	8.5	4.5	3.5	2.9	2.3
2.............	20.2	9.7	7.2[a]	6.0	4.9[c]
3.............	24	24	14.9[b]	11.6	9.1[d]
4.............	24	24	24	24	19.8
4.5.............	24	24	24	24	24

[a] In model test, pass was kept open 8 hours per day.   [b] In model test, pass was kept open 16 hours per day.
[c] In model test, pass was kept open 3.8 hours per day.   [d] In model test, pass was kept open 8 hours per day.

computed for different velocity limits in the canal. If the controlled water level can range the vertical distance between points E and F, then the period when the pass could remain open is the horizontal distance along the time scale between points E and F, which will vary with the rate of rise of the natural

tide. This analytical indication of the relation between controlled velocity, natural tide range, and time of opening is summarized in Table 23, which shows hours per day when the navigable pass would be available for traffic.

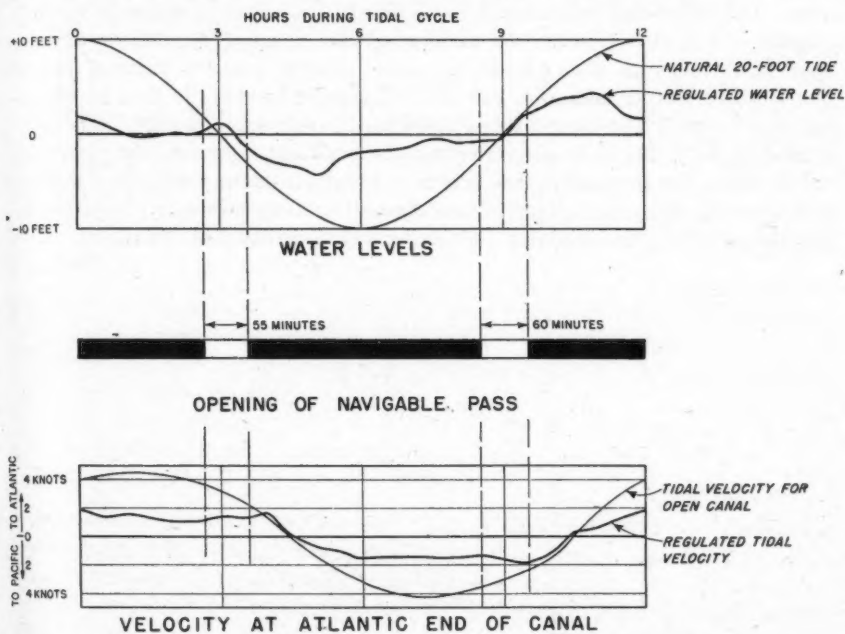

FIG. 39.—MODEL TEST; REGULATION TO 2 KNOTS, 20-FT PACIFIC TIDE
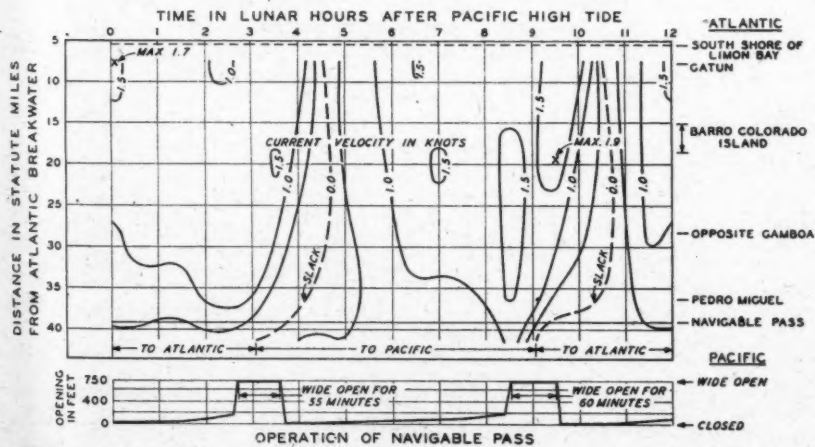

FIG. 40.—TIME-POSITION-VELOCITY DIAGRAM; 2-KNOT REGULATED CURRENT; 20-FT PACIFIC TIDE

Regulated Flow in Sea-Level Canal Model.—Fig. 39 shows one cycle of model test observations for current regulated to a maximum of 2 knots with a 20-ft

Pacific tide, for which the navigable pass remained open for one 55-min period and for one 60-min period during each 12 hours. In this test the navigable pass was used for flow regulation instead of the water-control structure. The regulated velocities are shown also in the time-position-velocity diagram of Fig. 40. Comparison with the similar diagram (Fig. 36) for unregulated tidal velocities shows much the same general velocity pattern, but of course with reduced maximum values. The model tests made thus far (October, 1947), which have also covered other tidal ranges and velocity limits as indicated in Table 23, have demonstrated the possibility of controlling currents and keeping the pass open for periods approximating the computed values. Additional testing and analysis of model results will be necessary, however, to provide data for gate schedules for routine daily operation of the canal.

SHIP PERFORMANCE IN RESTRICTED CHANNELS

By C. A. Lee[23] and C. E. Bowers,[24] Juniors, ASCE

Synopsis

The purpose of the restricted channel model tests at the David Taylor Model Basin was to obtain information which would be of assistance in the selection of the cross-sectional dimensions and in the design of bends for specified conditions of canal operation. The tests were sponsored by the Panama Canal under the authority of Public Law No. 280, Seventy-ninth Congress. The model studies included:

1. An investigation of the effect of varying the cross-sectional dimensions of the channel for both one-way and two-way traffic;
2. The comparative handling characteristics of several different types of ships under various conditions;
3. The effect of current in the channel on the handling characteristics of the ships; and
4. A comparison of several types of bends.

The first tests were conducted with a single ship in a straight channel and in still water. The experience gained from these tests is fundamental to a complete understanding of the more special problems which follow the description and the results of the straight channel, still-water, one-way traffic conditions.

TABLE 24.—General Information Concerning Models
Tested in Restricted Channels

Model No.	Scale ratio, λ	Dimensions of Model (Ft)			Dimensions of Full-Scale Ship (Ft)			Propellers		No. of rudders
		Length[a]	Beam	Draft	Length[a]	Beam	Draft	No.	Rotation	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
3769......	45	20.00	2.51	0.717	900	113	32.25	4	Outward	2
3850......	35	20.58	2.855	0.914	720.6	100	32.13	2	Outward	1
3992......	86	10.00	1.256	0.4025	860	108	34.625	4	Outward	2
3748-4....	45	10.01	1.33	0.6326	455	59.84	24.48	1	Right hand	1
3673-A....	44.5	20.00	2.585	0.815	890	115	36.27	4	Outward	2
4018......	45	16.00	2.221	0.714	720.6	100	32.13	2	Outward	1

a Water-line length of loaded vessel.

It was thought that the two primary problems in the selection of canal characteristics were (*a*) the controllability of full-scale ships operating at selected speeds in the canal, and (*b*) the change of level of ships in the canal.

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The controllability of full-scale ships from model studies appeared to be the more difficult to evaluate and as a result the test program emphasized this part of the studies.

The models representing larger ships were selected for test on the basis of difficulty of handling in restricted channels. The model of a "Liberty" ship was selected for test as representing an average ship transiting the canal. A particular object of study was to discover the effects on the Liberty ship when meeting and passing the larger vessels. Table 24 gives general background information on the various ship models used during the investigation.

NOTATION

Referring to Fig. 41, let: b_y be the distance from the center line of the channel to the gravity center of the ship (point G); b_z be the distance between the
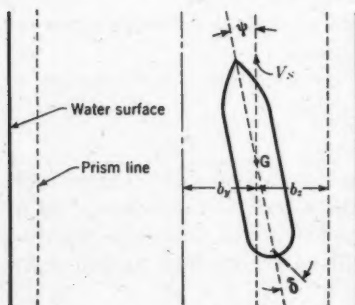

FIG. 41

gravity center of the ship and the prism line; V_c be the water speed; V_s be the ship speed (with respect to water); δ be the rudder angle; λ be the scale ratio; and ψ be the angle of yaw. In the restricted channel investigations, Froude's law is used as a basis for adjusting velocity, rate of change of rudder, and revolutions per minute on the propellers, to produce conditions similar to full-scale operations. Froude's law sets forth a nondimensional parameter which is used for model studies in which gravitational influences predominate. Another nondimensional parameter used to express the effect of viscosity on the flow about a vessel is the Reynolds number, **R**.

The model vessels and the models of the restricted channels were constructed geometrically similar for the various test conditions—that is, all linear dimensions were reduced directly in accordance with the selected scale ratios. Under these circumstances the flow characteristics (velocity and pressure) would be similar if the fluid properties were such that the Froude and Reynolds numbers, respectively, are equal for model and full-scale conditions in all cases. The Froude number, which allows for the influence of gravitational forces, may be defined as:

$$\mathbf{F} = \frac{V^2/a}{\gamma/\rho} \dots\dots\dots\dots\dots\dots\dots\dots(1)$$

The Reynolds number, which allows for the effect of viscosity, is

$$\mathbf{R} = \frac{V\,a}{\mu/\rho} \dots\dots\dots\dots\dots\dots\dots\dots(2)$$

in which V is the velocity, in feet per second; a is a convenient linear dimension, in feet, defining a fixed boundary condition; γ is the specific weight, in pounds

per cubic foot; ρ is the density in slugs per cubic foot; and μ is the dynamic viscosity, in pound-seconds per square foot.

The effects of viscosity will not be large enough to read by scale if the Reynolds number is so small that a large part of the boundary layer is laminar rather than turbulent. The actual size and scale ratios of the models used in this investigation insure that the Reynolds numbers are large enough to preclude the scale effects produced by viscosity. Consequently, it was only considered necessary to employ the same Froude number for the model as for the full-scale condition. Thus, the following relationships for time rates and velocity result:

(1) Time rates for the model (that is, rate of rudder movement, revolutions per minute of the propellers, rate of calling orders from "pilot" to "quartermaster," and any other time rates) should equal the time rates for the various elements on the full-scale ship multiplied by the square root of the linear ratio.

(2) Velocity for the model (that is, speed of the model ship and velocity of the water in the channel during moving water studies) should equal the velocity of the full-scale vessel divided by the square root of the linear ratio.

SHIP-HANDLING CONSIDERATIONS IN RESTRICTED CHANNELS

Navigation in restricted channels or canals is very difficult, not only because of the limited space available but also because of various hydrodynamic phenomena that introduce additional hazards. One of these phenomena is popularly referred to as "bank suction." Bank suction occurs when a vessel is closer to one side of a restricted channel than it is to the other side or when the vessel passes projections in the channel. Its effect is to cause the vessel to sheer or deviate from its original course. It could be described as an interaction between the ship and the channel boundaries. As a result of this effect, an asymmetrical flow distribution develops on the two sides of the vessel, creating unbalanced forces which tend to force the vessel off its original course. If the vessel is under way in a restricted channel, on a course parallel to but to one side of the center line of the channel, the water surface between the bow and the near bank will build up above the level of the normal water surface with the result that the bow is forced away from the near bank. As the water flows aft along both sides of the vessel to fill the void left by the stern, the level of the water surface drops below the normal surface level. The level of the water surface between the vessel and the near bank drops lower than the level on the other side, with the result that the stern of the vessel is forced toward the near bank. The net result of the difference in water level on the two sides of the vessel is to cause the ship to sheer away from the near bank. In some instances the sheer that results cannot be overcome by the rudder and the vessel strikes one of the banks. It is necessary to use a rudder setting which tends to turn the vessel toward the near bank to counteract this effect. Thus, if a large vessel were near the right bank, it would be necessary to use "right rudder" to counteract this effect. If it were desired to return to the center of the channel, the rudder could be eased off enough to allow the vessel to return slowly to the center. If, by using selected rudder angles, the heading of the vessel can be maintained parallel to the bank while

the vessel is off center in the channel, the resultant of all side forces acting on the vessel will be a force toward the near bank. If this heading is maintained for a time, the vessel will move bodily into the near bank. However, if the vessel is given a slight angle of yaw away from the near bank and a rudder angle just sufficient to counteract the yawing moment, both the side force and moment will be neutralized and the vessel will maintain a path parallel to the bank.

Another hydrodynamic phenomenon that may be serious during the transiting of vessels through restricted channels is the change of level of the vessel. When the vessel is under way in shallow water or in a restricted channel, the water surface in the vicinity of the vessel drops below the level of the normal water surface because of the increase in the velocity of the water as it flows around the vessel, and the vessel drops with it. If the initial draft of the vessel is quite large with respect to the depth of water in the channel, the ship may touch bottom if under way at relatively high speeds, whereas it would have ample clearance if it were stationary or traveling at a low speed. The magnitude of this change of level is a function of the ship speed, the dimensions and lines of the ship, and the channel dimensions.

Studies of One-Way Traffic

Relative to the straight-channel, one-way traffic, still-water studies of ship performance in restricted channels, primary emphasis was placed on an investigation of (*a*) the relative controllability of specified ships in channels of various cross-sectional dimensions, and (*b*) the change of level of ships in restricted channels. Another factor that may be of general interest, but which was omitted from this investigation, is the resistance of ships in restricted channels. In the past, many investigators have concerned themselves with the resistance of ships in shallow water, but few have treated the problem of ship resistance in restricted channels. Information on ship resistance in shallow water is available in several places.[25,26,27,28,29,30,31,32] Most available information on ship resistance in restricted channels appears in papers by G. S. Baker,[33] Francis Roubiliac,[34] F. E. Nelson,[35] and T. Izubuchis and S. N. Z. Kidkai,[36] although additional data on barges have been obtained.

[25] "Model Studies of Ship Motions in Canals," by Charles E. Bowers, David Taylor Model Basin, Washington, D. C., November, 1946.

[26] "Barge Canals—Dimensions," by J. M. Rankine, *Encyclopædia Brittanica*, 14th Ed., 1929, Vol. 4, pp. 720–727.

[27] "The Relation of Depth of Water to Speed and Power of Ships," *Engineering News*, Vol. 53, 1905, pp. 275–276.

[28] "Der Schiffswiderstand im bergrenzten Fahrwasser und sein Einfluss auf die Grössenverhältnisse der Schiffahrtskanäle" (On Ship's Resistance in Limited Waters and Its Effect on the Relationships Between the Sizes and the Channel), by M. Graevell, *Der Civil Ingenieur*, 1887, pp. 87–110.

[29] "The Influence of Depth of Water on the Resistance of Ships," by Charles P. Paulding, *Marine Engineering*, May, 1903, pp. 239–243.

[30] "The Resistance of Some Merchant Ship Types in Shallow Water," by Herbert C. Sadler, *Transactions*, Soc. of Naval Architects and Marine Engrs., Vol. 19, 1911, pp. 83–86.

[31] "Modell-Schleppversuche für Lastkähne im Kanalprofil" (Model Towing Tests for Barges in the Channel Profile), by Karl Schaffran, *Schiffbau*, Vol. 16, No. 13, 1914/1915, pp. 321–326.

[32] "A General Discussion of Resistance and Power Consumption of Ships in Different Depths of Water," by David W. Taylor, *Engineering News*, Vol. 53, 1905, pp. 276–279.

[33] "Steering of Ships in Shallow Water and Canals," by G. S. Baker, *Transactions*, Institution of Naval Architects, Vol. 66, 1924, pp. 319–340.

[34] "Speed of Canals," by Francis Roubiliac, *Minutes of Proceedings*, Inst. C. E., Vol. 76, 1884, pp. 160–265.

[35] "Handling Vessels in Restricted Waters," by F. E. Nelson, *Proceedings*, U. S. Naval Inst., June, 1928, pp. 446–456.

[36] "Experimental Investigations on Influence of Depth of Water Upon Resistance of Ships," by T. Izubuchis and S. N. Z. Kidkai, *Paper N4 Autumwerting*, Soc. of Naval Architects—Japan, 1937 (in Japanese).

An investigation of the controllability of ships in restricted channels is necessarily quite complex. It involves the effect of interaction between the vessels and the channel boundaries (bank suction), the steering characteristics of the vessel, and the effect of the restricted channel on the steering character- istics of the vessel. If an attempt is made to maneuver the model in a manner similar to the maneuvers of a full-scale vessel in a restricted channel, the "human element" or skill of the pilot becomes important. In an attempt to evaluate these factors, two general types of tests were set up. In each of these types it was planned to test several ship models in channels with various cross- sectional dimensions.

One type of test, the so-called observational tests, consisted of observing and photographing the models while they were under way and completely unrestrained in a restricted channel. The other type, the force-measurement tests, consisted of the measurement of side forces which developed when the model was held at various transverse positions in a stream of moving water. The side force and yawing moment were measured for various rudder angles and angles of yaw. In addition, the rudder angle required to overcome the turning moment caused by interaction was determined.

Observational Tests.—The observational tests were conducted in a Taylor Model Basin facility known as "the shallow water basin." This basin consists of a concrete-lined channel, 52 ft wide, 10 ft deep, and approximately 300 ft long. Fig. 42 is a photograph of the basin. The central part of the restricted channel is made of steel sections with adjustable sides, so that the angle of slope can be set at 18°, 30°, 45°, or 90° to the horizontal. The width of the channel can be varied by moving one or both sides along the basin floor. The depth of water is varied by changing the water level in the basin. The wooden slat structures outside the channel, and at the near end, are arranged to break up waves and surges set up by motion of the model. . The ship model, shown at the far end of the channel in Fig. 42, is operated by distant control from the special platform under the towing carriage. The water depth can be set at any value up to 10 ft. A towing carriage, which spans the basin, can be run in either direction at speeds up to about 8 knots. The functions of the towing carriage are (1) to tow models that are being tested for resistance or other per- formance characteristics; (2) to provide a movable observational and photo- graphic platform, and power supply, for studies of self-propelled models; and (3) to accelerate self-propelled models to the desired speed in a short time. The two steel walls were placed on the floor of this basin to form a smaller channel. The over-all length of this channel was 180 ft, which is equivalent to approximately 1.5 miles of full-scale channel. Its width could be varied from 0 to 23 ft. The side slope of the walls could be varied from an angle with the horizontal of from 18.25° to 90°. During the observational phase of the tests, the walls were set at an angle of 45°. Fig. 43 is a photograph of a model running through a restricted channel 60 ft deep and 300 ft wide with side walls set at 45°. The "pilot" issues orders to the helmsman who steers the self- propelled model by the remote-control rudder gear located on the lower plat- form of the towing carriage. The flexible cable at the stern carries the electrical leads and is held in position by the movable boom. The reflector mounted
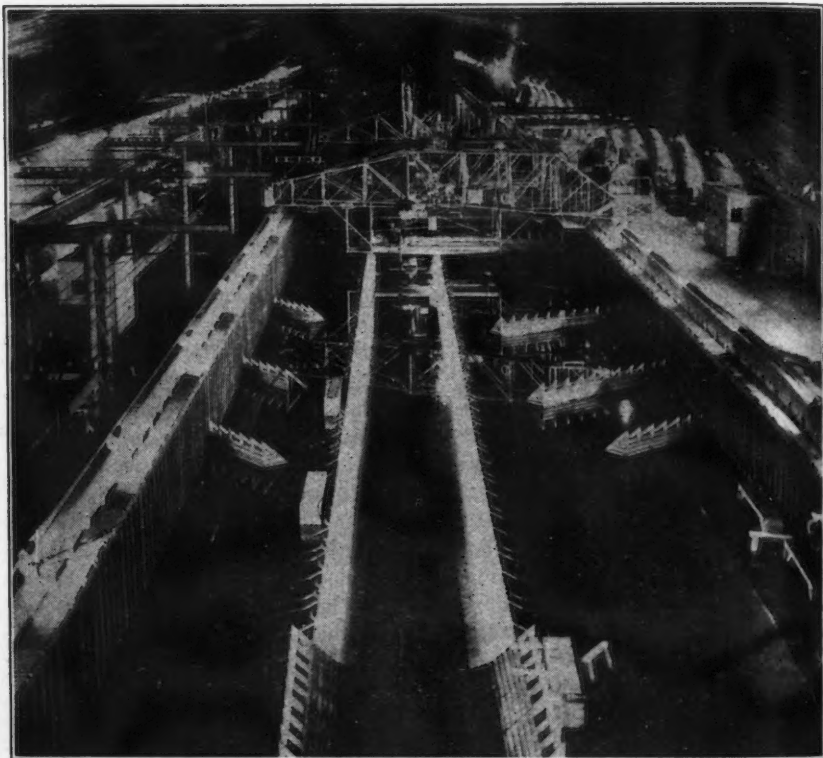
FIG. 42.—GENERAL VIEW OF BASIN BUILDING, SHOWING THE RESTRICTED
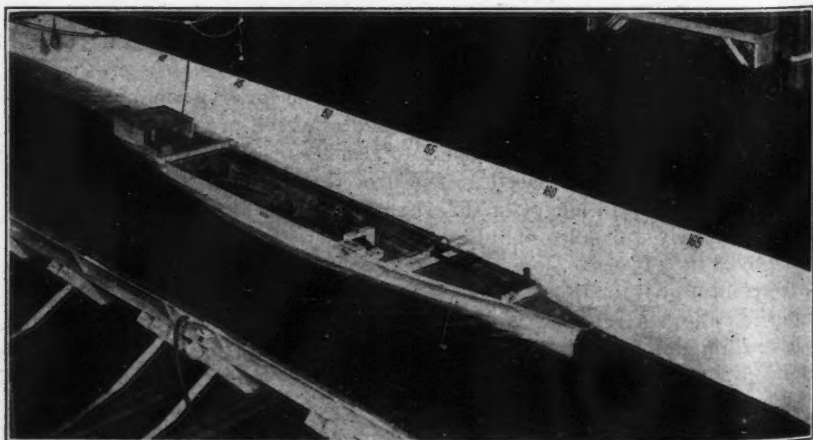CHANNEL SETUP IN THE SHALLOW WATER BASIN

FIG. 43.—MODEL 3769 UNDER WAY IN A STRAIGHT CHANNEL

near the bow casts a beam of light on a horizontal scale at the far end of the channel, thus giving the pilot a good indication of changes in the ship's heading.

The tests in this shallow water basin were conducted using Taylor Model Basin models 3769, 3748-4, and 4018; and they represented channels of 300 ft, 500 ft, and 700 ft wide at the bottom with a 1:1 side-wall slope. Each width was tested with depths of water of 45 ft, 60 ft, and 80 ft. The scale ratio for the model studies was 1:45. On the basis of Froude's law, the model speed during the tests was equal to the full-scale speed divided by the square root of 45. For example, the model speed corresponding to 10 knots full scale was 10 divided by 6.71, or 1.49 knots.

Two types of observational tests were conducted. One type (referred to as the "rudder release tests") was for the purpose of obtaining a comparison of the bank suction or interaction for various off-center positions of the ship in the channels. The nature of the tests was such that the magnitude of the yawing moment, caused by interaction between the ship and the channel boundaries, was obtained in terms of the rudder angle required to counteract it.

Some question may be raised with regard to the advisability of expressing the yawing moment in terms of rudder angle rather than in a more orthodox form such as foot-pounds. One advantage of this method is that it expresses the moment in a term which is familiar to most people who are acquainted with the handling of ships. On the average ship the maximum rudder angle that can be used is approximately 35°. Thus, if the rudder angle required to counteract the bank suction or interaction, for some specified condition, is from 25° to 30°, it is obvious that the moment is quite large with respect to the maximum counteracting moment that can be developed by the rudder. A disadvantage of this method, in some instances, is that the turning moment developed by the rudder may not be directly proportional to the rudder angle at large rudder angles.

In conducting the first type of observational tests, the model was attached to the towing carriage by two pins which held the longitudinal axis of the model parallel to the center line of the model canal channel. The towing carriage was then accelerated to the desired speed and at the same time the speed of the propellers of the model was increased to that which would propel the model at the desired speed. The model was then released from all contacts with the towing carriage, with the exception of a light flexible cable that supplied power to the propeller motors. The path and heading of the model were observed from the towing carriage. A light source on the bow of the model cast a 2-in. beam of light along an extension of the model center line to a horizontal scale at the end of the channel. By observing the movement of the beam, the observer could note, instantly, the changes in the heading of the model. It should be noted that the purpose of the towing carriage was to accelerate the model in as short a distance as possible, thus providing the maximum length of the channel available for observation of the unrestrained model. The model was attached to the towing carriage on a line parallel but to one side of the center line. If the rudder had previously been set at zero, it would be noted that the model sheered away from the near wall on release from the carriage. As soon as this sheer was observed, the model and carriage were

stopped and the model was again attached to the carriage at the same off-center position. Before releasing it a second time the rudder would then be set at an angle which would normally turn the model toward the near bank. If the model again sheered away from the near bank, it would be returned to the carriage and the rudder angle would be increased. This procedure was repeated until a rudder angle was selected which would just counteract the yawing moment caused by the interaction. After this rudder angle had been selected, the procedure was repeated at several higher speeds. The model was then moved to a point farther off center and the complete procedure was repeated. In this manner, data were obtained for the rudder angles required to counteract interaction at various off-center positions, various ship speeds, and for channels with various widths and depths.
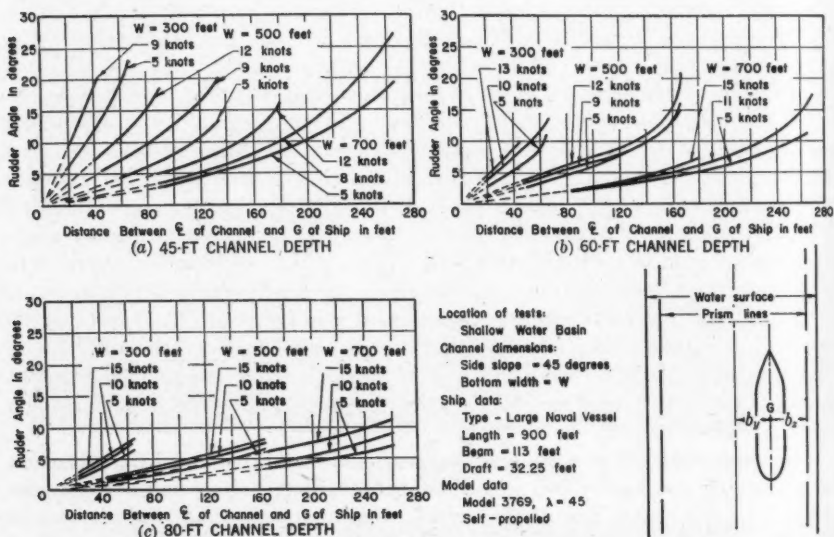

FIG. 44.—RUDDER ANGLE FOR EQUILIBRIUM AS A FUNCTION OF THE DISTANCE BETWEEN THE CENTER LINE OF THE CHANNEL AND THE CENTER OF GRAVITY (POINT G) OF THE SHIP

Fig. 44 is a plot of the rudder angle required to counteract interaction or bank suction against distance between the center line of the channel and the center of gravity of the ship. Specifically, these rudder angles are required to counteract the yawing moment that exists when self-propelled model 3769 is released parallel to, and at various distances from, the center line of the channel. Data for several channel widths and ship speeds have been plotted on each graph for purposes of comparison. For a given speed the distance that the ship can navigate off center in the channel without exceeding a specified rudder angle may be selected for each channel width. It may be noted from these data that the rudder angle required to counteract the effects of interaction is especially dependent on the ship speed at a 45-ft depth. Furthermore, the slope of these curves, or the rate at which the rudder angle increases, is much

steeper at the shallower depths and narrower widths. In most instances the rudder angle for equilibrium was measured for at least three off-center positions of the ship. These positions were varied for each width of channel so as to cover the proper range of rudder angles. For selected channel widths and ship speeds, the curves are indicative of the effect of channel depth on the
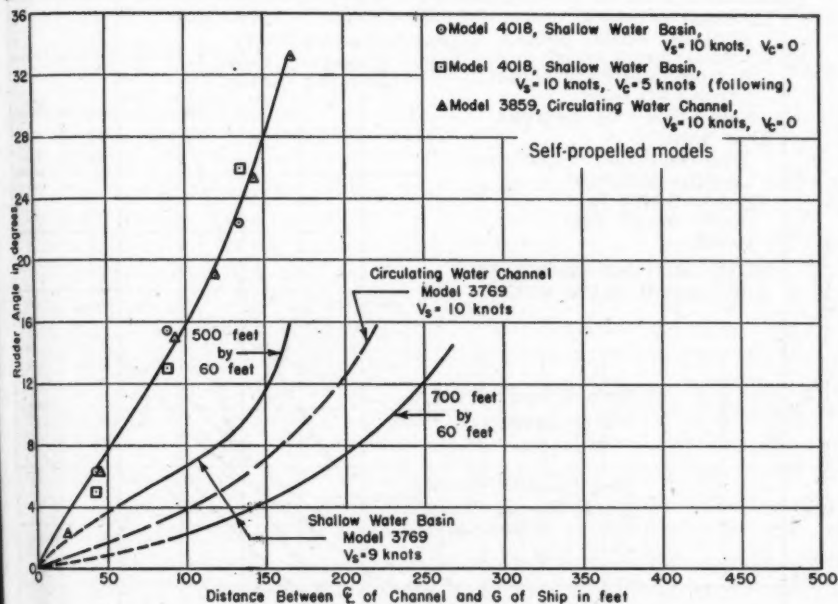
	MODEL 3769	MODEL 3859	MODEL 4018	MODEL 3769
Location of tests:	Circulating Water Channel	Circulating Water Channel	Shallow Water Basin	Shallow Water Basin
Channel dimensions:				
Width =	600 feet	500 feet	500 feet	500, 700 feet
Depth =	60 feet	60 feet	60 feet	60 feet
Side Slope =	Vertical	Vertical	45 degrees	45 degrees

FIG. 45.—RUDDER ANGLE FOR EQUILIBRIUM AS A FUNCTION OF THE DISTANCE BETWEEN THE CENTER LINE OF THE CHANNEL AND THE CENTER OF GRAVITY (POINT G) OF THE SHIP

yawing moment caused by interaction. It may be noted that, for the 300-ft width, the yawing moment caused by interaction at a channel depth of 60 ft is less than half as great, in terms of rudder angles, as it is for the 45-ft depth. An increase in depth from 60 ft to 80 ft causes a further decrease in the required rudder angle but the additional change is much smaller. For channel widths of 500 ft and 700 ft, there is a similar decrease in the required rudder angle with increasing depth.

Subsequently, additional tests were conducted on the model of the naval vessel as well as on models of a large tanker and a Liberty ship. Some of these tests were conducted in moving water for the purpose of checking the effect of

currents in the channel on the interaction between the ship and the channel boundaries. Others were conducted in the circulating water channel for the purpose of comparing results between that facility and the shallow water basin. The results of some of these tests are presented in Figs. 45 and 46.
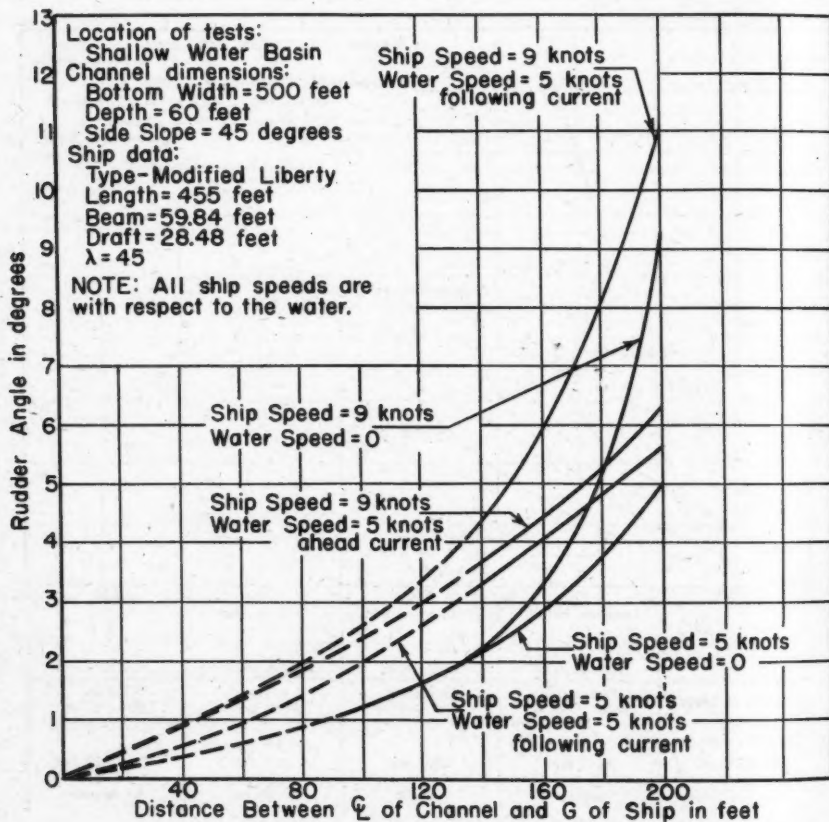

FIG. 46.—RUDDER ANGLES FOR EQUILIBRIUM AS A FUNCTION OF THE DISTANCE BETWEEN THE CENTER LINE OF THE CHANNEL AND THE CENTER OF GRAVITY (POINT G) OF THE SHIP

With reference to Fig. 45, the data for model 3859 (tanker) in the circulating water channel were obtained by holding the model stationary at various transverse positions in a moving stream of water. The test methods are described in the section entitled "Force-Measurement Tests." Similar data were obtained in the shallow water basin, by the methods just described, for both still water and currents. The data indicate that the two facilities produce nearly identical results. Also, the interaction between the ship and the channel boundaries is apparently the same for both still and moving water at the same ship speed with respect to the water.

In Fig. 46, data are shown for the rudder angles required to counteract interaction for a Liberty ship in both still and moving water. (These data are based on self-propelled tests of model 3748-4. The rudder angles are required to counteract the yawing moment that develops when the ship is nearer one wall then the other.) For the same ship speed with respect to the water, the curves indicate that the rudder angles are dependent on the current. However, the maximum difference between the curves for practical operating conditions is less than 3°, which is not considered significant in view of the limited amount of data obtained on this model.

A second type of observational test was based on maneuvering a self-propelled model by remote control. Throughout the tests an effort was made to duplicate full-scale operating conditions as closely as possible. In these tests, the model, when accelerated to the desired speed, was released from all contacts with the towing carriage, with the exception of a light flexible cable which supplied power to the propeller and rudder motors. By closing a double-throw switch on the towing carriage, the rudder on the model could be moved to any desired setting. This setting could be altered as frequently as desired throughout the run. A large indicator was mounted on the stern of the model which showed the instantaneous rudder angle at all times. In addition, the rudder angle was indicated by a "selsyn" system on the model control panel of the towing carriage. By observing the movement of the beam of light from the bow, the observer or pilot could note, instantly, any changes in the heading of the model. During the run, the pilot attempted to maintain the model on a course parallel to or on the center line of the channel. On orders from the pilot, an assistant manipulated the controls which actuated the rudder. Soon after the model was released from the carriage, it was photographed at intervals of about 1 sec by an overhead camera. The camera provided a record of the path of the model, its speed, and the rudder angles used.

During this phase of the tests, an attempt was made to maintain a course parallel to, and at a specified distance from, the center line, as well as directly on the center line of the channel. This procedure differs somewhat from actual full-scale operating conditions in that the pilots usually attempt to stay on the center line of a canal except when meeting another ship. However, it was thought that the off-center runs might provide additional information. Maneuvering runs were taken at about three transverse positions in channels with bottom widths of 300 ft and 500 ft and with depths of 45 ft, 60 ft, and 80 ft. An attempt was also made to conduct separate runs for various speeds ranging from 5 knots to 15 knots, but in numerous instances it was not possible to reach the top speed of 15 knots because of excessive change of level of the model or because of other hazardous operating conditions.

During this phase of the tests, it was noted that for a specified width the depth of water was very important with regard to ease of handling. At the 60-ft and 80-ft depths, it was much easier to control the model and to maintain a course parallel to the bank at greater distances off center than was possible at the 45-ft depths. This condition was especially noticeable at channel widths of 300 ft and 500 ft.

During these tests (which were made to find the maximum off-center distances at which specified ship models could be satisfactorily maneuvered), certain phenomena were allowed to develop and the resultant action of the vessel was observed. It was noted that, in any off-center position in the channel, if the bow was allowed to swing a few degrees away from the near bank there was a sudden drop in the water surface between the stern of the vessel and the near bank. As a result of these conditions, a moment was produced which tended to increase the rate of swing of the bow. If the condition was not corrected immediately, the vessel would develop such a sheer that application of rudder would not bring the vessel into a condition of equilibrium. As a result, the vessel would sheer across the channel and the bow would strike the far bank or the stern would be forced into the near bank. If the heading of the model was maintained exactly parallel to the near bank during an off-center run, by using the correct rudder angle, the model would drift laterally toward the near bank. As it moved closer to the wall, the yawing moment became larger and it was necessary to increase the rudder angle to maintain the heading of the model parallel to the bank. Eventually the model would ground unless the yawing moment became so large that a sheer developed. Therefore, the angular position of the vessel in the channel, especially in an off-center position, is important. Considerable judgment is required of the pilot in balancing the forces acting on the bow and stern by application of rudder and slight changes in heading. It should be understood that application of rudder alone will not always bring the vessel into equilibrium for an off-center position in a restricted channel but that a proper amount of heading must also be maintained.

When the total of all the rudder angles used for a given run were averaged arithmetically and plotted against the average distance between the center line of the channel and the center of gravity of the ship during the run, the resultant curves closely approximated the rudder angles for equilibrium which were obtained in the first part of the observational tests. The primary value of these data is that they substantiate the equilibrium rudder angles which were derived in the first part of the test, as an indication of the yawing moment caused by interaction.

The maneuvering data are still being analyzed and it is hoped that additional quantitative information on controllability may be obtained from them. The preceding results are based on only one model and are intended primarily to indicate some of the general characteristics of the interaction or bank suction between a ship and the channel boundaries. It would undoubtedly be desirable to obtain information on the effect of varying the cross-sectional dimensions of the ship, holding the length constant, as well as information on a variety of ship types and designs. However, a program of this type would be too extensive for the present investigations.

Force-Measurement Tests.—The purpose of the force-measurement tests was to supplement the observational tests in order to evaluate the various factors affecting the controllability of ships in restricted channels. As the term implies, the tests consisted of measuring the forces resulting from interaction between the ship and the channel boundaries. The tests were conducted in a

Taylor Model Basin facility known as the circulating water channel. This facility is similar in principle to a wind tunnel. It consists of a test section 22 ft wide, 9 ft deep, and 60 ft long, a return channel beneath which the water passes from the downstream end of the test section back to the entrance of the test section, pumps in the downstream vertical leg to force the water around this circuit, and related equipment. The water depth in the channel can be varied from 0 to 9 ft. Movable walls may be placed in the test section to vary the width of the channel. The water speed in the channel may be varied as desired up to 10 knots.

The basic difference between these tests and the observational tests is that the model can be held stationary while the water flows past, whereas in the observational tests the water is stationary and the model is moved. The advantage of this type of facility lies in the fact that readings can be taken continuously for as long a period as is desired, whereas, in the towing basin, the length of time available for the observation of a given run is limited by the length of the channel. It should be noted that the force-measurement tests are static tests and involve a different method of analysis than do the observational tests, which are of the dynamic type. In the force-measurement tests, the model was restrained while the forces that tended to make it move were measured. It is recognized that this is an artificial condition, with respect to a normal operating condition for the full-scale ship, but the data obtained provide the background for an analysis of the dynamic tests.

The models were tested at various off-center positions in channels with various cross-sectional dimensions and at the equivalent full-scale speeds of from 4.5 knots to 10 knots. During the tests the model was attached to the dynamometer in the desired position by three arms which extend downward from the dynamometer. Two of the arms were 2.5 ft forward and 2.5 ft aft, respectively, of the center of gravity of the model. These two arms measured the side force acting on the model and restrained any movement laterally. The third arm, referred to as the drag arm, was located forward of the other two and measured any fore and aft force acting on the model. The dynamometer was designed so that the model can be attached at any desired transverse position in the channel. When it is desired to give the model an angle of yaw, the entire dynamometer is turned. Thus, all forces acting on the model are measured parallel or perpendicular to the center line of the model regardless of its angle of yaw.

As previously discussed, when a ship is off center in a restricted channel, the side forces acting on it tend to make it sheer away from the near wall. In the observational tests, the rudder angle required to counteract the yawing moment caused by these forces was determined by trial and error which involved releasing the model at a specified distance from the center line and trying various rudder angles until equilibrium was obtained. It was also pointed out that, in addition to the rudder angle, it was necessary to give a slight angle of yaw away from the near wall to produce a true condition of equilibrium. In the force-measurement tests, equilibrium rudder angles were determined by measuring forces on the model at various angles of yaw. This result was accomplished by varying the rudder angle and the angle of yaw until

both the yawing moment and the side force became zero. The model was self-propelled and the speed of the propellers was varied until the drag of the model was equalized. Fig. 47 is a graph of the rudder angle required for equilibrium plotted against the distance between the center line of the channel and the center of gravity of the ship. (If the ship is near the right bank, it is neces-
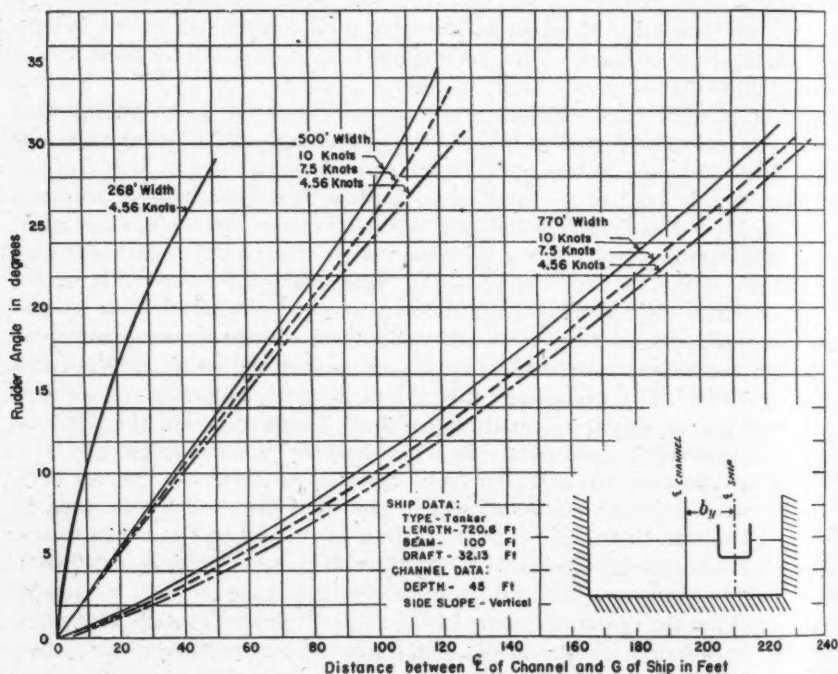

FIG. 47.—RUDDER ANGLES NECESSARY TO PRODUCE A CONDITION OF EQUILIBRIUM
FOR VARIOUS DISTANCES BETWEEN THE CENTER LINE OF A RESTRICTED
CHANNEL AND THE CENTER OF GRAVITY OF A SHIP

sary to use "right rudder" and to yaw the ship to port to counteract the moment and side force that develop. The reverse would be true if the ship were near the left bank.) The data are for a large tanker in channels with widths of 268 ft, 500 ft, and 770 ft. The side walls of the channel were vertical.

In addition to determining the rudder angle and the angle of yaw required for equilibrium, measurements were taken of the side force and yawing moment that developed when the model was held parallel to the wall and at various transverse positions in the circulating water channel. These tests were conducted for the purpose of determining the magnitude of the interaction or bank suction in terms of force and moment as opposed to the preceding data which indicate the magnitude of these effects in terms of the rudder angle and the angle of yaw required to counteract them. The yawing moment and the side force were plotted with respect to the distance between the center line of the channel and the center of gravity of the ship. Fig. 48 is a comparison of the

side force and yawing moment acting on the tanker (length 720.6 ft and beam 100 ft) at various transverse positions in channels with three different widths (268 ft, 500 ft, and 770 ft) for one depth (45 ft) and for one ship speed (4.56 knots). The scale ratio was 3.5. The data in Fig. 48 are based on self-propelled tests of model 3859. They are plotted for the condition with the ship to starboard of channel center line. The rudder angle was set at 0° when the yawing moment and the lateral force were measured. The rudder angle required to counteract the yawing moment of zero yaw has also been plotted for comparison. During these tests the rudder angle and the angle of yaw were set at 0°. The rudder angles required to counteract the yawing moment were
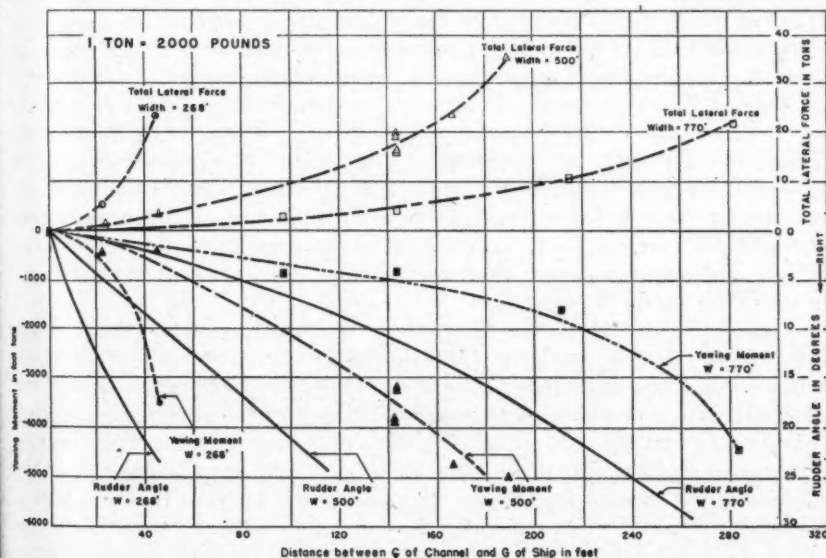

Fig. 48.—Yawing Moment and Lateral Force Acting on a Ship That Is Parallel to, and at Various Distances from, the Center Line of a Restricted Channel

measured, following the measurement of side force and yawing moment. Previous tests had indicated that there was a slight difference between the true rudder angle for equillibrium and the rudder angle to produce zero moment at zero yaw—thus necessitating separate measurements for the equilibrium condition. The rudder angle for zero moment at zero yaw has been plotted in Fig. 48 for comparison with the moment curves.

   Summary of Observation and Force-Measurement Tests.—According to the original test program for the investigation of ship performance in restricted channels, models of two different ships were to be tested under identical conditions during the straight-channel, one-way traffic studies. The two models selected for the tests were a large naval vessel and a large twin-screw, single-rudder tanker. In accordance with this program, both models would have been subjected to both observational and force-measurement tests. At the

request of The Panama Canal, the observational studies on the tanker were deferred until a later date to expedite other phases of the studies.

In both the observational and force-measurement tests, the rudder angle required to counteract the yawing moment caused by the interaction between the ship and the channel boundaries has been determined. Rudder angle is believed to be the most practical term for use in the study of the controllability of ships in restricted channels. The magnitude of the required rudder angle is a function of the channel dimensions, the size, ship lines, rudder and propeller characteristics, ship speed, and the position of the ship in the channel. Thus, the rudder required by two different ships for a specified off-center position will be affected by the steering characteristics of the ships. The study of rudder angles required for many different ships would be valuable for purposes of ship design and for the operation of ships in restricted channels. However, for the immediate purpose of indicating the necessary size and proportions of a canal channel, it is believed that study of the models of the large naval vessel and of the large tanker should provide sufficient information. The smaller and easier-handling ships should not present a problem in channels which are designed to handle the large ships. The naval vessel was originally selected for study in these tests because of its great size, although its steering characteristics are excellent. The tanker was selected as being representative of a type of large twin-screw, single-rudder ships whose steering characteristics in restricted waters are quite poor.

During the present studies, the models were tested in channels with a considerable variation in width and depth. The data have been plotted to facilitate a comparison of the effect of width and depth on the magnitude of the effects caused by interaction between the ship and the channel boundaries. It is thought that the rudder angles required to counteract these effects provide a comparison of the actual difficulty the pilot might have in controlling the ship under the various conditions. In general, it is believed that a condition requiring the use of a relatively high rudder angle would be dangerous for the ship. Available data on the relationship of yawing moment to rudder angle are quite limited.

Change-of-Level Test.—When a vessel is under way in still water, it is found that the water ahead of the vessel moves forward, outward, and downward. At a comparatively short distance aft of the bow the forward motion ceases, but the water still moves outward and downward to make way for the body of the vessel. Near this point the water starts to flow aft. This negative flow continues to within a short distance of the stern where the water closing in and upward behind a vessel has a forward motion. Wherever changes in velocity or direction of flow occur, there is usually a change in the elevation of the water surface. In shallow water or canals, the region of disturbed water about the ship is confined to a much smaller area than in water of unlimited width and depth. Thus, in a restricted channel, the reverse flow past the vessel has a greater velocity for the same ship speed. The net result is a larger change in the elevation of the water surface, with respect to the normal water surface, in the vicinity of the ship. In the past, considerable study has been devoted to the variation in the resistance of ships in shallow water and restricted chan-

nels, but very little work has been done on the change of level of ships under these conditions. Studies of ship resistance in shallow water and restricted channels are described extensively elsewhere.[25,27,28,30,32,36,37,38,39]   In 1904, Henry N. Babcock[40] described a series of measurements that was taken on change of level of ships that were under way in shallow channels. Measurements were taken from shore with the use of a surveyor's level. He reported a change of level of 4 ft for one of the ships tested. Some additional information on the change of level of model or full-scale ships under way in shallow water or canals has been presented elsewhere by Mr. Bowers.[25]

Fig. 49 is a plot of some actual measurements, during two independent tests, taken on the elevation of the water surface of a model. The lowering of
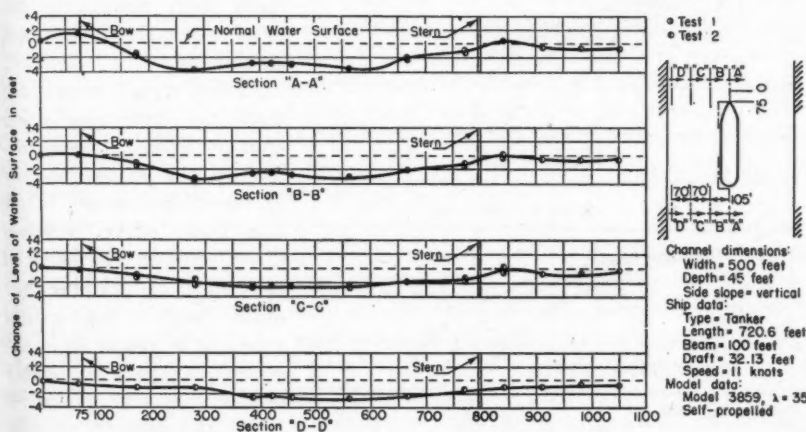

FIG. 49.—WATER-SURFACE PROFILE AT VARIOUS LONGITUDINAL SECTIONS FOR
A SHIP ON THE CENTER LINE OF A RESTRICTED CHANNEL

the water surface in the vicinity of the ship causes a corresponding lowering of the ship, with the result that it may ground in an area where the normal water depth is in excess of the draft of the ship. In addition to an increase in the change of level of a ship in restricted waters, as compared to deep water, it has been found that there is also a considerable increase in ship resistance in restricted waters. For shallow water of unlimited width, the increase in resistance depends on the depth of water. In a restricted channel the resistance is also a function of the channel width.

As a part of the Taylor Model Basin investigation of ship performance in restricted channels, the change of level of model 3769 was measured for a range of ship speeds from 5 knots to 15 knots, full scale, in channels with the cross-

[37] "Tidal Currents and Their Effect on Navigation," by J. A. Conwell, Special Eng. Div., The Panama Canal, Diablo Heights, Canal Zone, 1941.

[38] "Tests on the Wall Interference and Depth Effect in the Royal Aeronautical Experimental Seaplane Tank and Scale Effect Tests on Hulls of Three Sizes," by L. P. Coombes, W. G. A. Perring, V. W. Battle, and L. Johnston, *Technical Report*, Aeronautical Research Committee, Vol. 2, 1934–1935.

[39] "The Effect of Size of Towing Tank on Model Resistance," by John P. Comstock and C. H. Hancock, *Transactions*, Soc. of Naval Architects and Marine Engrs., Vol. 50, 1942, pp. 149–197.

[40] "Some Model Experiments on Suction of Vessels," by David W. Taylor, *ibid.*, Vol. 17, 1909, pp. 1–21.

sectional dimensions previously given. It was not possible to reach a speed equivalent to 15 knots in some instances due to excessive change of level which caused the model to touch the bottom of the channel. In these tests the model was attached to the towing carriage by connections that permitted a small amount of fore and aft movement and also allowed the model to trim freely. Indicators, which were mounted on the carriage, were attached to the model near the bow and stern by a cable arrangement so that vertical movements of the bow and stern could be read on large dials. The carriage was accelerated to the desired test speed and at the same time the propeller revolution was increased until the model was fully self-propelled. Readings were then taken of the level of the bow and stern with respect to the level when the model was stationary.

From tests it was noted that the bow and stern change level at approximately the same rate up to the critical speed, where the bow curve reverses direction, and the slope of the stern curve becomes quite large. The critical speed of a vessel in a restricted channel is roughly defined as the speed at which the relative velocity between the ship and the reverse flow past the beam of the ship is equal to the speed at which the relative velocity between the ship and the reverse flow past the beam of the ship is equal to the speed of the wave of translation for that particular depth of water. Near the critical speed, the water-surface level around the ship changes quite rapidly with any sli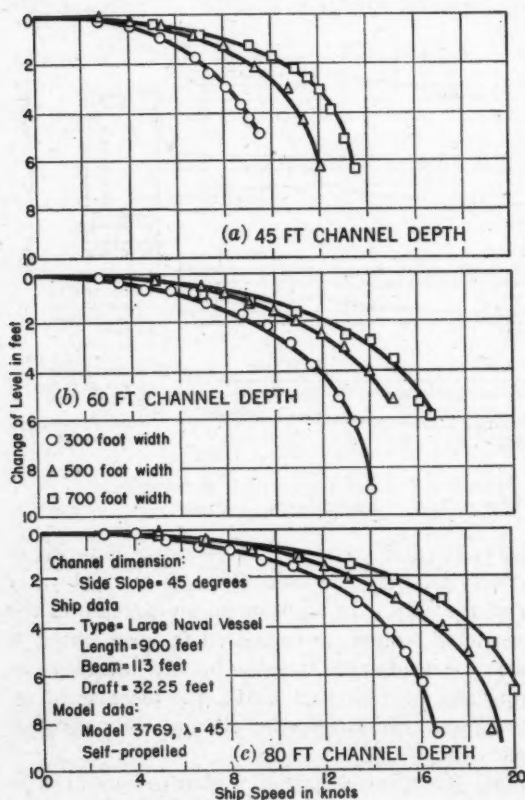ght change in relative speed. The models were not tested at speeds above the critical because of possible damage to the model and because there is little possibiltiy of operating the full-scale ship under these conditions, even at the greater depths where there is no danger of grounding. For a specified channel, the change of level of the ship varies approximately as the square of the velocity.

FIG. 50.—EFFECT OF CHANNEL WIDTH ON THE CHANGE OF LEVEL OF A SHIP ON THE CENTER LINE OF A RESTRICTED CHANNEL

In Fig. 50 the data have been plotted to illustrate the variation in change of level with variations in the width and depth of the channel. (The curves shown are for the stern only. The bow curves are similar.) A large change of level would probably be indicative of excess bank wash and unstable handling conditions due to interaction or bank suction.

The effect of the transverse position of the ship in the channel on its change of level was also investigated. Measurements were taken for various off-center positions of the model for channel widths equivalent to 300 ft and 500 ft and for channel depths equivalent to 45 ft, 60 ft, and 80 ft. The differences recorded for these tests were not of sufficient magnitude to present and would not be determining factors in the selection of channel dimensions.

To determine the effect of the propellers on the change of level of the ship, the model without propellers was towed on several occasions and the results compared with those obtained from the self-propelled tests. In the towed tests the shape of the curves was quite similar to that in the self-propelled tests. The bow and stern curves had the same relationship as in the self-propelled tests, and the critical speed was approximately the same. However, for most of the speed range up to the critical, the towed tests indicated from 8% to 15% less change of level than did the self-propelled tests.

Discussion of Change-of-Level Tests.—The change of level of a ship under way in restricted waters is of importance in the present investigation because (a) at relatively high ship speeds, the ship may ground due to excessive change of level; (b) a large change of level is indicative of the formation of large waves which may result in serious bank wash; and (c) at subcritical speeds a large change of level is indicative of a high ship resistance as compared to the resistance at the same speeds in deep water. The change of level of a ship under way in a restricted channel is a function of the dimensions and lines of the ship, the ship speed, the cross-sectional dimensions of the channel, and the position of the ship in the channel.

In general, an increase in ship speed produces an increase in the change of level at all speeds up to the critical. For a specified channel, the change of level of a ship increases approximately as the square of the speed for sub-critical speeds. The exact relationship between the change of level of the ship and the ship speed is apparently dependent on the cross-sectional area of the channel. At speeeds above the critical, an increase in ship speed may result in a reduction in the change of level of the ship. During the present investigation, the range of ship speeds used in the tests did not exceed the critical speed.

STUDIES OF TWO-WAY TRAFFIC

The purpose of the two-way traffic studies was to obtain information that would be of assistance in the selection of the cross-sectional dimensions of a channel adequate for the meeting of two ships of specified types. As part of this study it was desired to obtain information on the interaction between the two ships and between the ships and the channel boundaries.

Prior to this phase of the studies it was decided by The Panama Canal that two-way traffic studies be based primarily on the meeting of a large naval vessel

and a Liberty ship. It was further requested that the tests be conducted for: (1) Still water and currents of 3 knots and 5 knots, (2) ship speeds up to 10 knots with respect to the water, and (3) various widths and depths of channel.

When two ships are meeting each other in a restricted channel, each ship interferes with the lines of flow about the other vessel with the result that asymmetrical pressures develop on the two sides of each vessel, tending to divert the ship from its original path. Throughout the maneuver, the turning moment caused by these pressures changes direction several times. For example, as two vessels approach each other, the pilots maneuver them out of the center of the channel. When the bows of the two vessels are almost directly opposite each other there is a tendency for the water surface to build up between the bows forcing them apart, or causing the vessels to yaw away from each other. As the vessels draw abreast of each other, the bow of each vessel tends to move toward the low water surface in the vicinity of the stern of the other, with the result that they yaw toward each other. When the sterns are almost directly opposite each other, there is a tendency for the sterns to move toward each other, thus reversing the direction of yaw. Superimposed on these effects (which result from interaction between the two vessels) is the effect caused by interaction between each vessel and the channel boundaries. In general, this latter effect tends to cause the vessel to yaw away from the near bank. Thus, the maneuvering of the vessels is affected not only by the size, speed, and paths of the vessels but also by the cross-sectional dimensions of the channel.

At the conclusion of most of the straight-channel, one-way traffic studies, certain general criteria were established by The Panama Canal. The decision was made to establish 60 ft, tentatively, as a reasonable depth to be used in further studies of width as related to two-way traffic and bend studies. Drawing upon the experience of well-qualified Panama Canal pilots and Cape Cod Canal pilots, it was decided to establish, tentatively, a reasonable average rudder angle for the maneuvering of a vessel off center in a restricted channel. This average rudder angle would then be used to determine a safe width of canal. Because the vessels vary in size, form characteristics, and rudder power, in addition to the fact that every pilot will maneuver differently, it seemed advisable to set the average rudder angle for equilibrium as low as possible to provide for ample reserve rudder in case of emergency. Pilots considered that a 5° rudder angle was reasonable.
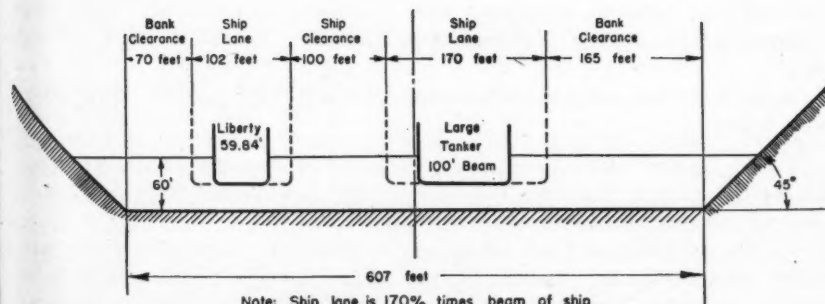
For example, in selecting a width of channel, the following procedure could be used: Fig. 51 shows two cross sections of a channel as determined from model studies, with outlines of the midship sections of (*a*) the large naval vessel and Liberty ship and (*b*) the large tanker and Liberty ship. The dotted outlines represent the average ship lane of the vessels, as determined from observational studies previously mentioned. The widths of the ship lane are approximately 170% of the beam of the vessel for a channel 500 ft wide and 60 ft deep, at a ship speed of 10 knots. The distances of the ships from the prism lines were computed from Figs. 44 to 47 for a rudder angle of 5°, and for a ship speed of from 9 knots to 10 knots. The distance between the vessels was selected as the beam of the larger vessel. This fact was later confirmed with model

studies. The curves of Fig. 46 are based on rather limited data and it was thought desirable to select the maximum curve of rudder angle required, which was for the 9-knot ship speed with a following current of 5 knots.

The sum of the various distances from Fig. 51 gives a channel width of 595 ft for the large naval vessel and the Liberty ship, and a width of 607 ft for the
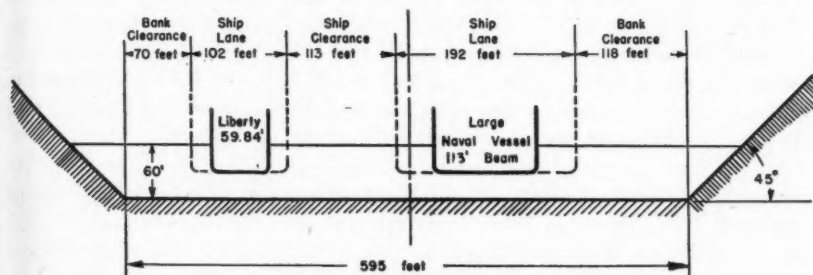
(a) PASSING CONDITIONS FOR LARGE NAVAL AND LIBERTY SHIP

(b) PASSING CONDITIONS FOR LARGE TANKER AND LIBERTY SHIP

FIG. 51.—PROPOSED WIDTHS FOR TWO-WAY TRAFFIC

large tanker and the Liberty ship, thus presenting one method of analyzing the data. Other selected rudder angles and channel depths will give widths in accordance with the selected values. It should be emphasized that such assumptions will only vary the distance between the ship and the near bank. The widths of the ship lanes and the distance between vessels offer reasonable values for this specific problem. Other combinations of vessels would vary these values. The data in Fig. 51 do not constitute a design intended for the Panama Canal. Other considerations may be necessary in the selection of proper width and depth. The sixth paper in this Symposium will develop this phase of the problem as applied to the Panama Canal.

Model studies were then conducted with the Liberty ship meeting the large naval vessel in a restricted channel with still water and with ahead and following

currents of 3 knots and 5 knots. During the tests one model was towed by an endless cable in one direction while the other model was maneuvered by remote control in the opposite direction. Observations of the handling characteristics and the effect of interaction between the two vessels were limited to the maneuvered model.

With the exception of a limited number of tests at a channel width of 700 ft, all two-way traffic studies were conducted in a channel with a bottom width equivalent to 500 ft and a depth equivalent to 60 ft. It was intended to conduct further tests with a width equivalent to 600 ft but these were deferred to pursue urgent bend studies. The major part of the studies was conducted with both ships traveling at the same speed with respect to the water.

All conclusions were based on visual observations of the models by the Panama Canal pilots (who operated the models) and by the Taylor Model Basin staff. During these tests it was attempted to simulate full-scale conditions as closely as possible. However, it was not possible to satisfy all conditions. The walls of the model channel were quite smooth as compared with the probable irregularities of full-scale channels. Also, the "pilot" had to think and act much faster during the model tests than would be possible on the full-scale ship, because of the time relationship between the model and the full-scale ship. However, the pilots who observed and participated in the tests believed that full-scale conditions were simulated very well and that, where differences did exist, the model was more difficult to control than the full-scale ship.

Subject to the preceding discussion, the following general observations were made:

a. It appeared that interaction between the two vessels did not cause any serious difficulty in handling, in a channel 500 ft wide and 60 ft deep, at the ship and current speeds employed.

b. The current speed did not appear to affect the magnitude of the interaction between the two vessels, for the same ship speeds with respect to the water. However, it would probably have an effect on the handling characteristics of the vessels in the vicinity of bends or large irregularities in the walls of the channel.

c. The Liberty ship steered best when it had a speed with respect to the water of 7.5 knots. A minimum speed at which good control could be maintained was approximately 5 knots.

d. The transverse position of the naval vessel had very little effect on the maneuvering of the Liberty ship as long as the clear distance between the two vessels while passing was at least 100 ft. For more general conditions it was thought that the distance should be at least 100 ft or a distance equal to the beam of the larger vessel—whichever was the larger.

BEND STUDIES

The primary purpose of the bend studies was to investigate the comparative difficulty encountered in maneuvering a specified ship around variously designed bends at a 1:86 scale ratio. It was also desired to investigate the

effects of variations in ship speed and current on the handling characteristics of a specified ship. Later in the investigation it was decided to conduct tests at a 1:45 scale ratio to study the maneuvering characteristics of various vessels and scale effects. Models 3769, 4018, and 3748-4 were used for the 1:45 scale-ratio studies and model 3992 was used for the 1:86 scale-ratio tests. The characteristics of the models are given in Fig. 41.

For the 1:86 scale-ratio tests, model 3992 representing a large naval vessel was selected and constructed and the movable walls were adjusted to the de-sired width and proper curva-ture for each test. A section of straight channel was con-structed ahead of the bend to permit accelerating the model before it reached the bend. The model was attached to the towing carriage while it was being accelerated, after which it was released from the carriage and maneuvered around the bend by remote control. Veteran Panama Canal pilots issued the helm and engine orders during each run. The control panel, par-allel light beam, and other

FIG. 52.—CURVES TESTED IN MODEL STUDIES

features were similar to those used in the two-way traffic studies and to part of the one-way traffic studies.

Tests were conducted on four types of bends at a 1:86 scale ratio, as shown in Fig. 52:

    (a) A 26° parallel bend;
    (b) A 26° parallel widened bend;
    (c) A 40° widened bend; and
    (d) A double 13° bend;

—and on two types at a 1:45 scale ratio:

    (e) A model of La Pita bend in the Panama Canal, as generally shown in Fig. 53; and
    (f) A 26° parallel bend, as shown in Fig. 52.

In connection with test (e), Fig. 53 shows a northbound transit of the *U. S. S. Wisconsin* through La Pita bend and the transit of a model of a large naval vessel through a 1:45 model of the same bend. The curves show the path of a point that is 0.455% of the ship length from the forward perpendicular and is on the center line of the vessel.

The tests were conducted at ship speeds of 5 knots, 7.5 knots, and 10 knots through the water, with both ahead and following currents of 0 knot, 3 knots, and 5 knots. When the bend studies were initiated, it was planned to in-
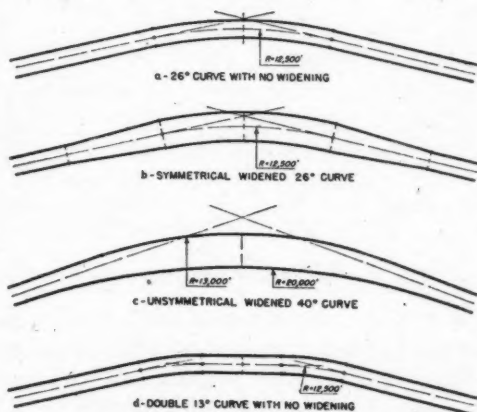
vestigate arbitrary bend angles of 20° and 40° and to use a straight-channel section at the entrance and exit of the bend with a width of 600 ft and a depth of 60 ft. However, at the conclusion of the first 40° bend test, available information indicated that a channel width of 560 ft (600 ft wide at the 40-ft depth), a depth of 60 ft, and a maximum bend angle of 26° would be of primary interest.

Notes

The bend shown is copied from the Annual Survey Chart, July 1, 1945, published by The Panama Canal, Department of Operation and Maintenance, Dredging Division, Gamboa, Canal Zone Panama Canal pilots endeavored to steer the self-propelled, remote controlled model on the most desirable course

Legend

Dashed curve represents path of Model 3769 (run H4-11)

Solid curve represents path of USS WISCONSIN

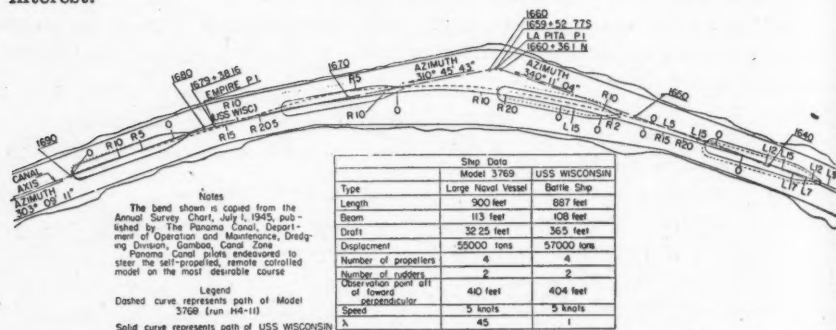|	Ship Data	
	Model 3769	USS WISCONSIN
Type	Large Naval Vessel	Battle Ship
Length	900 feet	887 feet
Beam	113 feet	108 feet
Draft	32.25 feet	365 feet
Displacement	55000 tons	57000 tons
Number of propellers	4	4
Number of rudders	2	2
Observation point aft of foward perpendicular	410 feet	404 feet
Speed	5 knots	5 knots
λ	45	1

FIG. 53.—COMPARATIVE SHIP PERFORMANCE IN MODEL AND FULL-SCALE LA PITA BENDS

At first it was felt desirable to attempt to maneuver the vessel both at the center line and at the quarter point on either side of the center line for all conditions of ship speeds and currents—in an attempt to indicate the desirability of meeting and passing a Liberty ship and a large vessel in the curved section. Although a substantial number of such tests were conducted for all the bend designs, the results indicated that such a maneuver might be dangerous if one vessel were large; but little difficulty was foreseen for Liberty ships meeting and passing in the bend. Therefore, the condition of a large vessel meeting and passing another vessel in the bend was eliminated as one of the requirements for the bend design at the present time. Further study may very well give a solution to this problem.

The comparisons of the 1:86 scale-ratio bends were then made for center-line transits only. Table 25 is a summary of the average maximum deviation from the center line of the channel for the various bends tested. ("Deviation" may be defined as the maximum departure of the forward or after perpendicular from the intended path of the center of gravity of the vessel. Width of path is the sum of the maximum port and starboard deviations plus the beam of the vessel for a particular test run.) Referring to Table 25, Panama Canal pilots endeavored to steer the self-propelled, remote-control models along the channel center line of the shallow water basin. Both models represented large naval vessels, with general dimensions as follows:

Dimension	Model 3769	Model 3992
Length, in feet	900	860
Beam, in feet	113	108
Draft, in feet	32.25	34.62

Model 3769 was tested in a 26° parallel bend ($\lambda = 45$); and model 3992 was tested in the other four bends ($\lambda = 86$). All measured data were taken from

streak photographs of the ship's path. The average width of path in Table 25 is based on the total number of runs made under the same test conditions. The average maximum deviation of the ship's path to port or to starboard of the channel center line is based on the total number of runs made under the same test conditions. Included in Table 25 are the average widths of path for the same conditions. The width of path included the beam of the vessel. All runs that were considered as satisfactory test runs were included in the average. Some runs were excluded from the analysis because of failure of the test equipment. The average values of rudder angles or average maximum rudder angles were not used as criteria because of the variations in the techniques used by the pilots in handling the vessel. Some pilots tended to use rather large rudder angles for short periods whereas others tended to use relatively small angles for longer periods of time. Also, various pilots favored one side of the channel more than the other, which will give an even larger variation in rudder angles due to bank interaction.

The first reaction of an observer is that a widened bend would provide more room for maneuvering than would a uniform width of channel in the bend. Actually, the widened bend studies indicated that maneuvering was more difficult, the actions of the vessel were more erratic, and the widths of path were generally greater, as indicated in Table 25. It is considered that this result is principally due to the continual variations in the forces resulting from bank interaction as a vessel transits through a gradually widening section. In the normal transiting of a vessel down a straight channel, it is very difficult to keep it positioned exactly on the center line. In transiting a bend, this task is even more difficult, especially with a large vessel, because it forms a tangent or secant to the curvature of the bend. In either of the latter two cases the vessel will be positioned slightly off center and some rudder angle will be required to prevent a sheering action from developing. Some force or rudder action is usually required to turn the vessel around the bend, although in some cases the bank interaction is such as to turn the vessel around the bend without the use of rudder action. However, if the interaction is too great, the vessel may swing too fast and develop a sheer in the same direction causing the vessel to strike the inner bank. For example, if a vessel enters a right-hand bend slightly off center to the left, the bank interaction is such that the bow will tend to swing around the bend; should the vessel swing too far, a sheer would develop that would require the left rudder to keep the vessel under control. However, if the vessel enters the right-hand bend off center to the right, a larger right rudder would be required to keep the vessel under control and also to turn it around the bend. In a parallel bend the forces created by bank interaction have nearly the same values as similar forces in a straight channel. Therefore, the pilot can judge the required rudder angle for equilibrium more readily than in a widened bend where the values of the forces are changed or are continually changing during the transit. Because the vessel is swinging slightly as it traverses the bend and is not "headed" on a straight course, this maneuver can best be made by constant attention to the swing of the vessel and by the "feel" of the condition.

TABLE 25.—Comparison... Ship...

Line No.	Angle	Description	Scale ratio	Total No. of center-line runs	Width of path[a]	Maximum Average Deviation (Ft) Port	Star-board[b]	Total No. of center-line runs	Width of path[a]	Maximum Average Deviation (Ft) Port	Star-board
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)

(a) Velocity of Ship in Relation to Current; V_e = 5 Knots

1	40°	Widened	86	2	360	59	193	2	763	468	187
2	26°	Parallel; widened	86	6	475	192	175	2	639	316	215
3	13°	Double	86	7	304	141	55	3	295	99	88
4	26°	Parallel	86	7	290	126	56	3	238	115	14
5	26°	Parallel	45	4	277	106	58	3	245	116	16

(b) Velocity of Ship in Relation to Current; V_e = 7.5 Knots

6	40°	Widened	86	6	466	166	194	5	643	439	337
7	26°	Parallel; widened	86	7	450	215	127	7	411	160	143
8	13°	Double	86	16	363	158	97	3	340	138	95
9	26°	Parallel	86	8	369	172	91	9	263	128	26
10	26°	Parallel	45	4	305	125	67	4	245	115	17

(c) Velocity of Ship in Relation to Current; V_e = 10 Knots

11	40°	Widened	86	2	723	337	278	1	401	244	49
12	26°	Parallel; widened	86	4	378	174	96	6	402	182	112
13	13°	Double	86	6	389	177	104	2	290	129	55
14	26°	Parallel	86	7	309	126	75	5	342	157	76
15	26°	Parallel	45	4	209	100	−4	4	231	124	−6

[a] Average width of path for all center-line runs.   [b] A minus sign in this colu...

The scale ratio used made it impossible to provide room for the pilot in the model and thus let him "feel" the action of the vessel as it deviated from its heading. A pilot on a full-scale vessel normally "feels" or anticipates the motion of the vessel well in advance of an observer not on the vessel. In addition, the scale ratios were so small that slight motions were difficult to detect from the carriage platform in time to make the proper corrections.

By considering the foregoing factors for the analysis of the 1:86 scale-ratio studies and by comparing the performance of the model vessel in the various bends, not attempting to extrapolate the values to full-scale operation, observers felt that the 26° parallel bend provided the best solution for all conditions of testing. This decision was reached by comparing similar runs of the various bends tested. For this comparison only one ship model was used. No appreciable differences were encountered in the 1:86 scale, 26° parallel bend for still-water or moving-water conditions.

One factor that should be mentioned in the case of moving water is that of variations in time of transit through a bend. No appreciable differences were noted in the model's action for the still-water and moving-water studies, pro-

SHIP PERFORMANCE

(11)	Ahead Current; $V_c = 5$				Following Current; $V_c = 3$				Following Current; $V_c = 5$				Line No.
Starboard	Total No. of centerline runs	Width of path	Maximum Average Deviation (Ft)		Total No. of centerline runs	Width of path	Maximum Average Deviation (Ft)		Total No. of centerline runs	Width of path	Maximum Average Deviation (Ft)		
			Port	Starboard			Port	Starboard			Port	Starboard	
(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	
(a) Velocity of Ship in Relation to Current; $V_s = 5$ Knots													
187	....	....	....	....	4	512	193	213	2	474	170	195	1
215	....	....	....	....	0	....	....	....	0	....	....	....	2
88	....	....	....	....	0	....	....	....	0	....	....	....	3
14	....	....	....	....	4	229	99	22	9	269	103	57	4
16	....	....	....	....	4	217	93	11	6	244	118	13	5
(b) Velocity of Ship in Relation to Current; $V_s = 7.5$ Knots													
337	7	649	337	204	2	622	214	300	3	342	94	141	6
143	4	430	185	138	0	....	....	....	0	....	....	....	7
95	6	406	175	123	0	....	....	....	0	....	....	....	8
26	7	297	147	41	4	235	80	46	5	255	107	40	9
17	5	257	92	52	4	183	91	−21	6	190	100	−23	10
(c) Velocity of Ship in Relation to Current; $V_s = 10$ Knots													
49	3	568	244	217	4	486	282	98	3	380	139	133	11
11	4	417	223	86	0	....	....	....	0	....	....	....	12
5	3	392	159	123	0	....	....	....	0	....	....	....	13
76	7	378	159	111	4	253	102	42	5	240	108	−23	14
−0	6	231	85	33	4	177	80	−16	6	197	101	−17	15

...ates that the center line of the ship was always to the port of the channel center line.

vided that the speed of the vessel with respect to the water remained constant. There were, of course, differences in time of transit. Appreciably fewer total rudder orders were required in transiting a bend with a following current than with an "ahead" current. However, the rudder orders necessary per unit of time were approximately equal for all conditions of testing in either still water or moving water, provided that the speed of the ship with respect to the water remained constant. Actually the vessel remains in the bend for a shorter period of time for the following current condition than for the ahead current condition. This fact usually gives the pilot the impression, from observations of the speed of the vessel over the ground rather than from observations of its speed through the water, that less time is available for adjusting his course during the transit of the bend—which evidently creates somewhat of a mental hazard for the pilots navigating through a bend in a following current. Experience proves that this condition is no more difficult than conditions of still water or ahead current.

At the conclusion of the 1:86 scale-ratio tests, it was decided to conduct two more bend studies at the larger scale ratio of 1:45:

a. The 26° parallel bend, shown in Fig. 52; and

b. The model of the La Pita bend, Panama Canal, as shown in Fig. 53.

The 1:45 scale ratio of the 26° parallel bend was tested:

a. To determine by comparison with the 1:86 scale ratio test if there were scale effect; and

b. To compare transits of models of other ships—namely, the Liberty ship and a large tanker.

The widths of path and deviations from the center line for the various test conditions are tabulated with the 1:86 scale-ratio tests in Table 25. Similar records were taken, by photographic methods, of all record runs. The results of this study indicate that some scale effect was present in the 1:86 scale-ratio studies. However, the actions of the vessel were quite similar. The scale effect was chiefly present in the additional time permitted the pilot for giving orders. Except for the poor handling quality of the large tanker, as determined in the straight-channel studies, no difficulty was experienced in transiting the 1:45 scale-ratio parallel bend with any of the models under any of the conditions as long as the transits were made on the center line. The Liberty ship model handled very well.

The model of the La Pita bend was tested at a 1:45 scale ratio in a further effort to determine scale effect by comparing model tests with a full-scale transit, as shown in Fig. 53. The model transit of the bend compares very favorably with the full-scale transit. However, studies of the Suez and Cape Cod canals indicated that bends of uniform width throughout were more desirable. In the case of the Suez Canal, a parallel bend had been widened to provide room for larger vessels. Ships which previously had had no difficulty began to experience some erratic action. Therefore, the bend was further widened in such a manner that it again became parallel.

The model studies have been limited to a small number of ships and bend designs. The comparison of widths of path for the 1:86 scale-ratio studies indicated that the 26° parallel bend would give the most satisfactory operating characteristics of the model for all conditions. This comparison was further substantiated by visual observations of the test runs, the number and magnitude of rudder orders, and the greater uniformity of results from pilot to pilot. The 1:45 scale-ratio studies of the 26° parallel bend showed improvement in the operation characteristics of the model over the 1:86 scale-ratio tests. This improvement could be attributed only to scale effect. The width of path in the bend compared quite favorably with the width of path for the straight channel when "crabbing" or "angular position" of the vessel in the channel bend is considered.

The 26° parallel bend was determined to be the most desirable bend of those studied and the test showed that currents did not produce any hazardous operating conditions. The model could be navigated as readily in moving water as in still water.

Summary

The variety of sizes, shapes, rudders, and propulsion characteristics of the many ships that will transit any channel is so great that only representative types from different general groupings could be used for the experiments. For these studies and their relation to the Panama Canal the vessels were selected because of (1) their large size, (2) their poor handling characteristics in restricted waters, or (3) their being representative of a large percentage of the present ships transiting the canal. It is felt that the models of the ships studied "bracket" these conditions and that the results obtained provide a guide for selecting the channel dimensions. Before a final selection of the channel dimensions is made, there are other factors, which could not be inserted into a laboratory investigation, to be studied and considered. Ships of the same class may not have similar steering qualities. The efficiency of operation of a vessel is measured to some extent by the age and the number of days the ship has been under way since overhaul. Seldom do two captains or pilots come to perfect agreement on the proper methods to use in maneuvering a vessel. Wind, rain, fog, and mechanical failures are other items that have to be considered. To a limited extent these factors can be offset by artificial aids to navigation such as range markers, buoys, and beacon lights, but there are always special conditions through which a ship must be navigated.

This paper has presented the information that is available from tests conducted at the David Taylor Model Basin. A study of the maneuverability and controllability of specified ships in specified restricted waters is a multifold problem. The results of the experiments and analysis thus far made give a more complete understanding of a problem which has harassed ship operators, engineers, and pilots for more than a century. A complete solution of the general problem has not yet been obtained; however, limits and trends have been established that may serve a useful purpose in determining (a) the speed at which vessels may operate in restricted waters, (b) the practical depth and width of a restricted channel, and (c) the actions that produce extraneous moments on the vessels. Generally the forces and moments acting on the various ship models were explained and the rudder angle required for controllability and balancing of those moments was used as a measure of the moment.

DESIGN OF CHANNEL

By J. E. Reeves[41] and E. H. Bourquard,[42] Assoc. Members, ASCE

Synopsis

Methods of design and analysis are presented in this paper for use in determining the minimum cross section of the channel and the alinement and treatment of curves in a sea-level canal at Panama, to provide safe and efficient transit of all vessels to the year 2000. The channel design is based on the operation of the canal as an open waterway with a maximum current of 4.5 knots. The design methods described in this paper utilize the marine operating experience of existing waterways and the results of ship model tests conducted at the David Taylor Model Basin, some of which have been presented in the preceding paper in this Symposium. A minimum channel section 600 ft wide, measured at a depth of 40 ft, and 60 ft deep is proposed.

Introduction

The existing Panama Canal has provided commercial and military shipping with a safe and efficient means of transiting the American Isthmus, since its opening in 1914. Almost 200,000 ships have passed through the canal and the total accident cost has been only $1,310,000—equivalent to $7.76 per transit. Any modification of the existing canal, or any new canal to provide the necessary national security, must also provide commerce with a waterway at least as safe and efficient as the existing canal. In a sea-level canal, the adequacy of the channel cross section and alinement are the primary considerations in accomplishing this objective.

This paper describes the methods of designing and selecting the sea-level canal channel. The channel of the sea-level canal referred to herein is the restricted part of the waterway as distinct from open reaches of harbors where the topography does not limit the width of the channel. Although this paper discusses primarily the sea-level canal, there is no essential difference in the method used in the design of either a sea-level or a lock canal channel except that tidal currents in the sea-level canal introduce a factor that must be weighed and compensated for in the design. A comparison of the final selection of the lock canal channel and the sea-level canal channel is included.

The importance of avoiding the construction of a channel larger than necessary is obvious when it is considered that the excavation for the sea-level canal would total more than 1,000,000,000 cu yd. On the other hand, prudent planning is required to meet all reasonable demands of navigation in the future to avoid expensive corrective work.

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[42] Chf. of Navigation Section, Special Eng. Div., The Panama Canal, Diablo Heights, Canal Zone.

Navigation Features of Existing and Sea-Level Channels

The channels of the existing canal are of three types: Those through the lake where the width and depth are unrestrictive to vessels; those through harbor or approach areas where both width and depth are restricted, but where channel banks are well below water level and interaction between ship and bank is insignificant; and those where both depth and width are fully restricted by the channel bottom and side banks. In the existing canal, the 8-mile reach north of Pedro Miguel Locks known as Gaillard Cut and a few miles of lock approach channels make up the full extent of restricted channel section. In this restricted section about 70% of all bank-striking and grounding accidents of the existing canal occur. The minimum section of the restricted channel has a bottom width of 300 ft and a minimum depth of about 42 ft. Fig. 54 shows this section with a battleship hull of the *Iowa* class in it. The

MINIMUM DEPTH, 42 FEET

MINIMUM WIDTH, 300 FEET

PRESENT PANAMA CANAL

40'

MINIMUM DEPTH, 60 FEET

MINIMUM WIDTH, 600 FEET, MEASURED AT DEPTH OF 40 FEET

PANAMA SEA-LEVEL CANAL

SCALE IN FEET
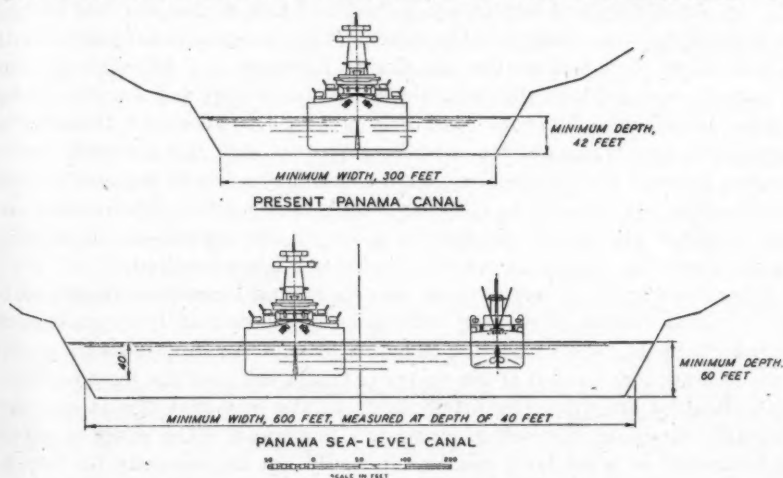
FIG. 54.—CHANNEL SECTIONS

sea-level canal, in comparison, would have approximately 30 miles of restricted channel section of minimum width extending from Limon Bay on the Atlantic side to the inner Balboa Harbor, Pacific side.

The existing canal is free of currents except for periodic tidal currents ranging up to a maximum of 1.5 knots in the sea approach channels. Currents in the sea-level canal would be limited by tidal regulation for the safety and convenience of shipping in the ordinary use of the canal to a maximum velocity tentatively established at 2 knots. If the tidal structures were damaged or wrecked by enemy action, the maximum current that would occur for short periods at extreme ranges of tide would be 4.5 knots. The importance of shipping in wartime requires that the channel be designed to provide safe and efficient transit when operating as an open waterway in which currents up to 4.5 knots would occur.

A survey of restricted waterways and canals in which currents are experienced discloses that currents do not significantly increase the problem of ship

handling if (1) the ship is adequately powered and ruddered so that it can maintain sufficient headway for good controllability, and (2) the channel section and alinement are satisfactory for navigation under slack-water conditions. However, mechanical failures and personal errors, even if they occur rarely, are essential considerations in analyzing the safety of navigation. The loss of control in a following current would probably result in accidents of greater severity and frequency, because of the shorter time available for regaining control and the greater speed of the ship on striking the bank. To reduce the likelihood of accidents under such circumstances, additional width of channel for maneuvering is desirable.

Climatic conditions, particularly fog, play an important part in operating and scheduling transit in the existing canal; however, they do not have a major bearing on fixing the channel dimensions, alinement, or treatment of curves.

Fogs are of frequent occurrence in Gaillard Cut during the late hours of the night in the rainy season. Operations in the existing canal are scheduled so that ships do not enter the cut during fog periods. Although no exact estimate is practicable of the probable extent, frequency, and duration of fogs in a sea-level canal, some increase in each may be expected. However, no operation in fogs is planned for either an improved lock or a sea-level canal at Panama because the daytime capacity would be sufficient to meet all conceivable demands of traffic to the year 2000. Electronic navigation aids have been perfected which make navigation in fog feasible, and these will be adaptable for use in the Panama sea-level canal if and when required.

Where strong winds are present, vessels having large superstructures, or lightly-loaded vessels, may have difficulties navigating at low speeds in restricted channels. This situation holds at present for large aircraft carriers at the lock approaches and at the curves in Gaillard Cut of the Panama Canal. The difficulties are primarily attributable to the fact that the carrier must necessarily approach the lock or curve at a low speed. The effect of wind is less important in a sea-level canal as it would not be necessary for ships to transit curves at low speeds and lockages would be eliminated or reduced drastically.

OPERATING CRITERIA FOR THE SEA-LEVEL CANAL

The general regulations under which the canal would be operated form an important part of the criteria for channel design. The essential criteria are those having to do with pilots, the meeting and passing of ships, ships' speeds, and interval between ships.

The design is based on the assumption that all transiting ships will be under the direction of experienced canal pilots. This condition exists in the present canal.

Single-direction traffic with alternating movements of traffic by direction would be feasible in a sea-level canal. This method of operation is used in the narrow and tortuous 8-mile Gaillard Cut for the transiting of large or poor-handling ships and for the transiting of all traffic in the cut after dark. Such a precaution unquestionably has been a major factor in the safety of transit through Gaillard Cut, and shipping has not been unduly delayed. The sea-

level canal, by contrast, would have about 30 miles of restricted channel. Since the average time of transit through the sea-level canal would be 4.5 hours as compared to 8 hours in the present canal, it would be possible to operate the new canal by limiting traffic to one direction and causing some additional waiting without loss of efficiency over present canal operation. However, it would be highly desirable to retain the benefit of the shortened transit time in the new waterway, and the imposition of the restriction of one-direction traffic on the efficient use of the canal in wartime is clearly inadvisable. Therefore the canal should be designed for two-way traffic.

A survey by the United States Navy revealed that increases in the size of ships may be expected during the remainder of the twentieth century, and that, for a lock canal, locks 200 ft wide and 1,500 ft long would be required. Whether ships approaching these dimensions will become a reality and will require transit through the canal is quite uncertain. In any case such exceptionally large ships will be few in number, so that the channel needs to be adequate only for the single-direction passage of these ships.

Any future Panama Canal must accommodate the largest ships of the Navy now afloat without restriction or delay, particularly in times of emergency. The transit of large naval vessels would not be required in both directions at the same time, and the canal would not be required to accommodate naval ships of the largest class traveling simultaneously in opposite directions. Likewise, the meeting of the largest of the existing commercial vessels, such as the *Queen Mary*, and large naval vessels need not be considered. The incidence of their meeting would be infrequent and resultant delays to avoid such meetings would be minor.

An index to the size of ships that would transit the canal with such frequency as to make special transiting arrangements inadvisable has been developed by frequency studies of transits of various size ships based on Panama Canal records. Table 26 is based on predicted traffic in the year 2000 and on the

TABLE 26.—THEORETICAL FREQUENCY OF MEETING OF SHIPS
IN THE RESTRICTED CHANNELS OF A PANAMA SEA-LEVEL
CANAL, IN THE YEAR 2000

Length of vessels (ft)	NUMBER OF TIMES PER DAY THAT THE VESSELS LISTED IN COL. 1 WOULD MEET AND PASS A VESSEL HAVING A LENGTH (FT) OF:		
	500 or more	400 or more	300 or more
(1)	(2)	(3)	(4)
More than 500...........	1	11	15
400 to 500..............	10	31	48

assumption that the canal will be operating 16 hours daily. It shows that two ships more than 500 ft long would meet about once a day. It would be impractical to design a canal for the meeting and the passing of two such ships, as the passing could be avoided with little, if any, delay by proper scheduling of transits. Vessels longer than 500 ft would meet vessels from 400 ft to 500 ft

long about ten times daily. Since special transit schedules to avoid those passings would cause individual delays of as much as 6 hours to 21 vessels on average days and to 31 vessels on peak-traffic days, the channel should be adequate for such passings. In using this criterion, "design ships" typical of each length group were selected. For the group, 500 ft and longer, a large loaded ore ship and a large naval vessel were selected. A loaded Liberty ship was selected to represent an average ship in the 400-ft to 500-ft class. Therefore, the channel of the sea-level canal is designed to permit a large loaded ore ship or a large naval vessel to pass a loaded Liberty ship with safety and ease. The controlling dimensions of these ships are shown in Table 27. The passing positions of a large naval vessel and a Liberty ship in the proposed sea-level canal channel are shown in Fig. 54.

TABLE 27.—CHARACTERISTICS OF DESIGN SHIPS

Description	DIMENSIONS (FT)				Displacement tonnage	Shaft horsepower	Horsepower per ton	Design speed (knots)	Controllability in proportion to size
	Overall length	Waterline length	Beam	Loaded draft					
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
Large naval vessel...	986	925	113	36	55,000	213,000	3.87	33+	Very good
Large ore ships......	583	574	78	35	32,450	13,000	0.40	17	Good
Liberty ship........	442	428	57	28	14,250	2,500	0.18	11	Good

Ships longer than 250 ft transiting the present Panama Canal are limited to speeds of 6 knots in the 300-ft channels, 10 knots in the 500-ft channels, 12 knots in the 800-ft channels, and 15 knots in the 1,000-ft channels, except in the case of the lock approaches or the channel through Balboa inner harbor where speeds are further reduced. Overtaking and passing is permitted only when one of the vessels is small or in the channels of Gatun Lake which are from 800 ft to 1,000 ft wide.

For the sea-level canal, a limiting speed of 10 knots was adopted for daily operation with the canal handling two-direction traffic. Naval task forces and convoys would be permitted a higher speed when not meeting opposing traffic. The minimum interval between ships would be about 1.5 miles, which is much greater than is normally required in canals. However, even at this interval, the capacity of the canal, with 16-hour daily operation, would be greatly in excess of the predicted peak-day traffic of the year 2000, so there would be no reason to reduce the interval.

Five criteria were used in the design of the sea-level canal channel, involving (a) pilots, (b) two-direction traffic, (c) single-direction traffic, (d) transit speed, and (e) ship interval. Specifically it was assumed that:

(a) All ships will carry a canal pilot.
(b) Naval ships up to the size of the largest existing naval vessel, and commercial vessels up to the size of a large loaded ore ship, will be permitted to meet and pass commercial vessels up to the size of a loaded Liberty ship.

(c) The design will be adequate for the clear channel transit of the largest ship that may be constructed to the year 2000.

(d) The normal speed of ships will be 10 knots; the passing speed, an average of 6 knots for both vessels.

(e) The average distance between ships will be 1.5 miles.

DESIGN METHODS AND INVESTIGATIONS

Previous Methods of Design.—Available information showed that, ordinarily, one of three methods has been used in the past to design navigable channels:

(1) One method is to provide a cross-sectional area of channel equal to a certain multiple of the immersed hull area of the largest ship expected to transit. The channel depth is obtained by allowing about 3 ft of clearance under the hull of this ship, plus an allowance for deposition of silt between dredgings. The channel width is determined from the previously selected channel area and depth.

(2) A second common method involves comparing the channels of existing waterways and, on the basis of operating records, selecting channel widths and depths for the proposed waterway. This method has also been used in selecting the treatment of curves.

(3) The third method employs empirical formulas developed from actual experience or from studies of the maneuvering characteristics of ships.

An analysis of navigation in various major world waterways showed that many factors not taken into account by these methods had an effect on channel requirements. For this reason, none of the foregoing methods was considered satisfactory for the design of a new canal at Panama.

Investigations for Present Studies.—The magnitude and importance of the present Isthmian Canal Studies led to an investigation of all aspects of channel design including many not previously considered. An outline of the investigations follows:

a. Examination of the literature on past practices in the design of restricted waterways;

b. Review of records of ship handling in the Panama Canal and consultation with pilots and others of the marine operating staff, The Panama Canal;

c. Compilation of physical data affecting channel design on the major waterways of the world (those having the greatest similarity to a sea-level canal at Panama being the present Panama Canal, the Suez Canal, the Cape Cod Canal, and the Houston (Tex.) Ship Channel);

d. Consultations with the pilots and operating personnel of the Suez Canal, the Cape Cod Canal, and the Houston Ship Channel on navigation in their respective canals and on the channel requirements for a sea-level canal at Panama, information also being obtained from ship operators regarding the adequacy and limitations of the major world waterways being studied;

e. Operation of ship models in restricted channels of different sizes under various conditions of current, by the Navy at the David Taylor Model Basin;

f. Construction and operation of a model of a Panama sea-level canal at a scale of 1:100, to determine the magnitude of uncontrolled tidal currents and

also the range of control that could be established if tidal regulation were adopted; and

g. Advice and recommendations from the Navy on features of the canal, particularly those affecting the requirements for ships operated by the Navy.

The results of these examinations and inquiries are not described in this paper except where they have an important bearing on specific problems of design.

DESIGN OF CHANNEL DEPTH

Both experience and ship model tests have established the fact that depth of section is of prime importance to good navigation, and that, within reasonable limits, the required width of channel is a function of the depth. Consequently, the depth of channel is established prior to the determination of the channel width.

The channel depth must be sufficient to allow transiting of the largest naval and commercial vessels expected to be constructed to the year 2000 under conditions of good controllability and without excessive squat or sinkage of the vessel when traveling at design speeds. Insufficient depth under a ship hull retards the flow of water to the propeller and to the rudder and reduces the effectiveness of both. The Navy has advised that a 50-ft depth over the sills of any new locks should be adequate for the largest naval or commercial vessels expected to the year 2000. As ships pass through the locks at very slow speeds, it is necessary that the channel have a minimum depth greater than 50 ft.

The depth of the channel at any location along the canal is measured below the water-surface profile connecting the low water elevation at each entrance to the canal.

TABLE 28.—RELATIONSHIP BETWEEN CHANNEL DEPTH AND SHIP DRAFT, FOR EXISTING WATERWAYS

Waterway	Depth channel (ft)	Transit speed (knots)	LARGEST VESSELS USING CANAL		LARGEST VESSEL HAVING GOOD CONTROLLABILITY	
			Draft (ft)	$\frac{Depth}{Draft}$	Draft (ft)	$\frac{Depth}{Draft}$
Gaillard Cut (Panama Canal)...	42	6	36	1.17	32	1.31
Suez Canal.................	43	7.5	34	1.26	28	1.54
Cape Cod Canal.............	40[a]	6–12	32	1.25	28	1.43

[a] Project depth is 32 ft, but scour has increased this depth to an estimated average of 40 ft.

A study was made of the relationship in certain major existing waterways between the channel depth and the draft of transiting vessels. The depth of the channels of three major waterways and the draft and the transit speeds of the largest ships normally using the waterways are given in Table 28. The draft shown for the "Largest Vessels Using Canal" is either that of the deepest draft vessel to transit the waterway or that of the maximum draft vessel that the canal authorities will permit to transit.

This study, which considered speed and channel width, indicated that for good vessel controllability the channel depth of the Panama sea-level canal should be about 1.5 times the draft of ships comprising the normal daily traffic, which in the year 2000 would probably include vessels having drafts up to about 40 ft. Consequently, a channel depth of approximately 60 ft is indicated. Study of the behavior of ships having a lesser ratio of depth to draft in existing waterways indicates that vessels of maximum size for the year 2000, with draft possibly approaching 50 ft, could safely transit a channel of 60-ft depth at the design speed of 10 knots.

The ship model tests discussed in the fifth paper of the Symposium clearly illustrated the importance of depth on the controllability of a vessel. The rudder angle required to control and hold a vessel parallel to the banks during off center-line maneuvers is the best indicator of relative controllability. Fig. 55 shows a curve, based on data obtained from the model tests, of the rudder
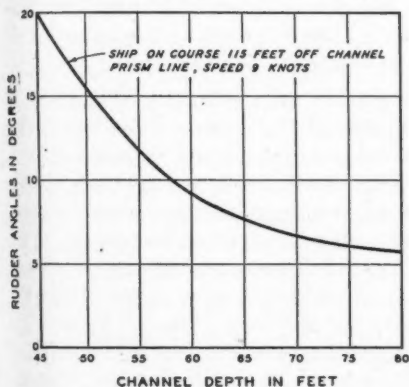
FIG. 55.—EFFECT OF CHANNEL DEPTH ON SHIP CONTROLLABILITY
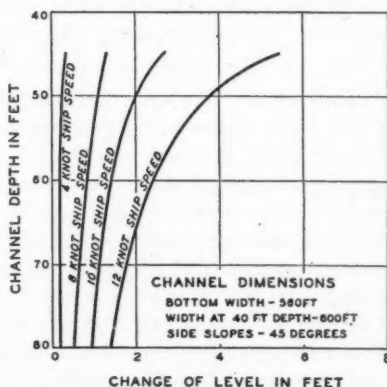
FIG. 56.—CHANNEL DEPTH VERSUS CHANGE OF LEVEL OF A SHIP

angles required to maintain a large naval vessel (length 900 ft, beam 113 ft, and draft 32.25 ft) on a course 115 ft off the prism line of a 500-ft wide channel (bottom width 500 ft with 45° bank slopes and depths ranging from 45 ft to 80 ft). A ship might be in such a position in a passing maneuver following momentary failure of equipment or wrong execution of a rudder command. At a 60-ft depth, a rudder angle of only 9° is required to control the vessel. In a 45-ft depth of channel, the angle required is 20° and the curve is rising rapidly, indicating that an unsafe condition is being approached where full available effort of the rudder would be required to control the vessel. The gain in reduction of the rudder angle as depth increased beyond 60 ft is slight and would not justify the increased construction costs.

In selecting channel depth, sufficient clearance must be provided between the ship and the channel bottom to avoid excessive sinkage or change of level of a vessel when transiting a restricted channel at the maximum allowable speed. Fig. 56 illustrates the effect of channel depth and ship speed on the change in level of the same large naval vessel as that in Fig. 55 in a channel with

the same width as the proposed sea-level canal channel.   The curves shown are
based on data obtained from ship model tests, verified in part by measurements
made on actual ships transiting the Panama Canal.   It was found that, with a
channel depth of 60 ft, sinkage would not be a factor so long as the channel has
a width of 300 ft or more.   The curves shown are for the stern only, the bow
curves being similar.   The ship traveled on the center line of the channel.

Based on the experience of existing waterways and experimental work at
the Taylor Model Basin, with due consideration of costs, a minimum depth of
60 ft for a sea-level canal is considered adequate.   For a lock canal having a
shorter length of restricted channel, a lower transit speed, and no currents, a
55-ft depth is considered adequate.

Design of Channel Width

Two methods were used in designing the width of channel, one (method *A*)
being based largely on the adjustments of similar existing waterways, and the
other (method *B*) being based primarily on observations of the behavior of
ships in existing waterways and in ship model experiments.

Method A.—The existing waterways used in the application of method A
should have characteristics somewhat similar to the waterway for which the
channel width is to be determined.   A study is then made to ascertain the
sizes of the largest ship or ships of average maneuverability that can easily
navigate each of the existing waterways under normal conditions with the de-
gree of safety and efficiency of transit desired in the proposed waterway.   The
selection of these ships may be on the basis of either single-direction or two-
direction traffic.   In those waterways where single-direction traffic is used as
the basis of analysis, the ship selected is called the "representative ship."   In
those where two-direction traffic is the basis of analysis, the larger of the two
passing ships is called "representative ship No. 1," and the smaller, "repre-
sentative ship No. 2."   The ships that are adopted as the maximum size for
which the proposed canal should be adequate to provide two-direction traffic
are designated "design ships" and the notations "No. 1" and "No. 2" apply to
the larger and smaller ones, respectively.   The channel section of each water-
way is then adjusted to be adequate for the "design ships" transiting the
adjusted waterway under the conditions that would exist in a sea-level canal.
This procedure requires four steps, as follows:

1. The channel dimensions of each existing waterway are multiplied by
the ratio of the dimensions of design ship No. 1 to those of representative
ship No. 1.   On the premise that the required channel section is directly pro-
portional to ship size, the resulting channels are then adequate for design ship
No. 1.

2. The adjusted channel sections of the existing waterways are now con-
verted to a common selected depth and to the same side slopes.   This is done
by holding the cross-sectional areas of the channel sections constant and by
adjusting the channel width.   The adjustment is based on the premise that
the navigational characteristics of a channel within reasonable limits are a
function of its cross-sectional area.

3. The third step is an adjustment for design ship No. 2.   Those channels which in the previous steps were based on single-direction traffic are increased in width by an amount equal to three times the beam of design ship No. 2. Channels based on two-direction traffic are increased (or decreased, if the first step resulted in representative ship No. 2 being larger than design ship No. 2) by an amount equal to five times the difference between the beam of design ship No. 2 and the beam of representative ship No. 2 after the latter has been multiplied by the ratio used in step 1.   The ratios of three and five are empirical and were developed from an analysis of the relationship of channel width to ships' beams.

4. The navigating and physical characteristics of each of the existing waterways are compared with those of the proposed sea-level canal and a final adjustment is made in the channel widths obtained in step 3.   The characteristics compared are traffic speed, currents, alinement, bank material, and length of restricted channel.

Application of Method *A*.—Channel studies using method *A* included the Cape Cod Canal, the Suez Canal, and the Houston Ship Channel, and Gaillard Cut of the Panama Canal.   The procedure followed in adjusting the Gaillard Cut channel of the present Panama Canal is described in detail.

The study of navigation in Gaillard Cut showed that for single-direction traffic the representative ship is a T-2 tanker, and for two-direction traffic representative ship No. 1 is a Liberty ship and representative ship No. 2 is a small ocean-going cargo vessel.   The *S. S. Pereira*, a vessel of good controllability with a beam of 32.7 ft and a draft of 15.1 ft, was selected as typical of small ocean-going cargo vessels.   Both cases are shown in Fig. 57 which illustrates the step-by-step procedure used in determining channel widths for Panama lock and sea-level canals.   The two-direction traffic condition which is shown as case 2 in Fig. 57 will be developed for a Panama sea-level canal in the following paragraphs.   In this application the large loaded ore ship (Table 27) is used as design ship No. 1 and the Liberty ship as design ship No .2.   One of the requirements of this method is that design ship No. 1 and representative ship No. 1 be of the same controllability classification, thus the large naval vessel (Table 27) could not be used.   The ratio of the beam of design ship No. 1 (78 ft) to the beam of representative ship No. 1 (57 ft) is 1.37 and the ratio of of drafts (35 to 28) is 1.25.   The average of the two ratios is 1.31 which, for purposes of conservatism, is increased to 1.35.   The actual minimum channel dimensions of Gaillard Cut and the dimensions after being multiplied by 1.35 in the first step are shown in Fig. 57.

Step 2 is a conversion of the Gaillard Cut adjusted section to the selected 60-ft depth and to 1-on-1 side slopes, which are approximately average bank slopes.   The bottom width is decreased to 363 ft as shown in Fig. 57.

Step 3 adjusts for design ship No. 2.   When the adjustment in step 1 was made, not only was the size of the channel increased, but representative ship No. 2 was also increased in size from a beam of 32.7 ft and a draft of 15.1 ft, to a beam of 44 ft and a draft of 20.5 ft, respectively.   The difference between the beam of design ship No. 2 and that of representative ship No. 2, after ad-
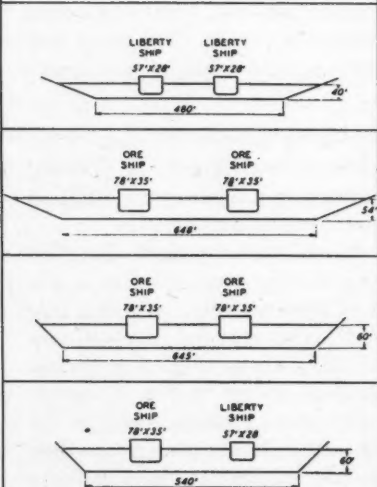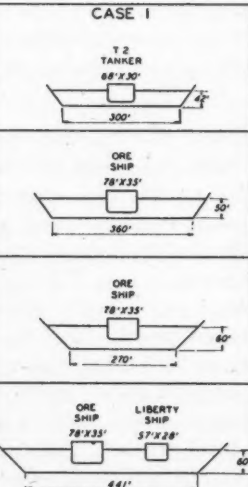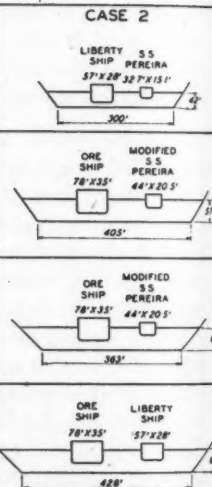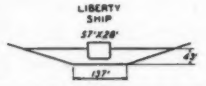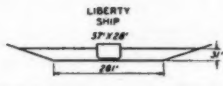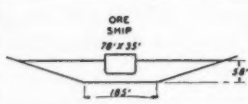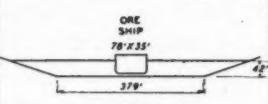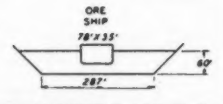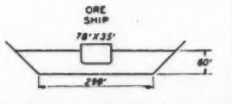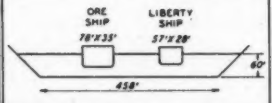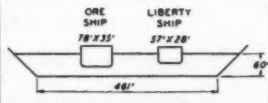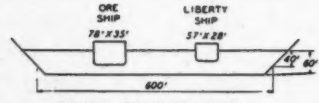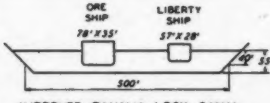
CAPE COD CANAL (LAND CUT CHANNEL)	GAILLARD CUT (THE PANAMA CANAL)	
77 MILES GRAVEL WITH BOULDERS UP TO 5.2 KNOTS AVERAGE ABOUT 9 KNOTS 75 DEGREES 7300 FEET 2.49 DEGREES CHANNEL DEPTH	8 MILES ROCK NO APPRECIABLE 6 KNOTS 30 DEGREES 5370 FEET 1.70 DEGREES LOCK DIMENSIONS	

CASE 1   CASE 2

LIBERTY SHIP 57'X28'   LIBERTY SHIP 57'X28'   40'   480'

T 2 TANKER 68'X30'   42'   300'

LIBERTY SHIP 57'X28'   SS PEREIRA 32'7"X15'   42'   300'

ORE SHIP 78'X35'   ORE SHIP 78'X35'   54'   648'

ORE SHIP 78'X35'   50'   360'

ORE SHIP 78'X35'   MODIFIED SS PEREIRA 44'X20.5'   57'   405'

ORE SHIP 78'X35'   ORE SHIP 78'X35'   60'   645'

ORE SHIP 78'X35'   60'   270'

ORE SHIP 78'X35'   MODIFIED SS PEREIRA 44'X20.5'   60'   363'

ORE SHIP 78'X35'   LIBERTY SHIP 57'X28'   60'   540'

ORE SHIP 78'X35'   LIBERTY SHIP 57'X28'   60'   441'

ORE SHIP 78'X35'   LIBERTY SHIP 57'X28'   60'   420'

SEA-LEVEL CANAL	LOCK CANAL	SEA-LEVEL CANAL	LOCK CANAL
(INCREASE) (DECREASE) DECREASE NO CHANGE INCREASE ABOUT RIGHT	(DECREASE) DECREASE (DECREASE) NO CHANGE (INCREASE) SHOULD BE APPRE- CIABLE LESS	INCREASE INCREASE DECREASE NO CHANGE INCREASE SHOULD BE APPRE- CIABLE MORE	(INCREASE) (DECREASE) (DECREASE) NO CHANGE (INCREASE) ABOUT RIGHT

WATERWAY CHARACTERISTICS	SEA-LEVEL CANAL	LOCK CANAL

LENGTH RESTRICTED CHANNEL
TYPE BANK MATERIAL
CURRENTS ENCOUNTERED BY SHIPS
TRANSIT SPEED WITH RESPECT
    TO LAND
LARGEST CURVE —ANGLE
            —RADIUS
ANGULARITY PER MILE OF
    RESTRICTED CHANNEL
MAXIMUM SHIP LIMITATIONS
	30 MILES
	MOSTLY OF ROCK
	4.5 KNOTS
	
	AVERAGE 10 KNOTS
	26 DEGREES
	12500 FEET
	
	3.4 DEGREES
	NONE TO YEAR 2000
		12 MILES (IN RESTRICTED CHANNELS)
		MOSTLY ROCK
		NO APPRECIABLE
		
		8 KNOTS
		33 DEGREES (IN RESTRICTED CHANNELS)
		6000 FEET
		
		1.00 DEGREES
		LOCK DIMENSIONS

FIG. 57.—DIAGRAM OF PROCEDURE FOR

DESCRIPTION OF STEPS	SUEZ CANAL (LAND CUT CHANNEL)	HOUSTON SHIP CHANNEL (250 FOOT PROJECT WIDTH CHANNEL)
WATERWAY CHARACTERISTICS LENGTH RESTRICTED CHANNEL TYPE BANK MATERIAL CURRENTS ENCOUNTERED BY SHIPS TRANSIT SPEED WITH RESPECT TO LAND LARGEST CURVE — ANGLE — RADIUS ANGULARITY PER MILE OF RESTRICTED CHANNEL MAXIMUM SHIP LIMITATIONS	75 MILES SAND UP TO 3 KNOTS 75 KNOTS 63 DEGREES 9966 FEET 33 DEGREES CURVES	125 MILES EARTH NO APPRECIABLE 4-8 KNOTS 109 DEGREES 3000 FEET 617 DEGREES CHANNEL DEPTH

SELECTED CHANNEL SECTIONS FOR PROPOSED PANAMA CANALS

SELECTION OF CHANNEL SECTIONS

justment by step 1, is $57 - 44 = 13$ ft. To adjust for design ship No. 2, the channel width is then increased by $5 \times 13 = 65$ ft. Thus, the bottom width of the Gaillard Cut channel section is changed from 363 ft to 428 ft.

Step 4 takes into account the difference in the navigating and physical characteristics of the waterways. Traffic speed in Gaillard Cut is 6 knots, but 10 knots is to be allowed in the Panama sea-level canal; therefore, an increase in the width of the Gaillard Cut section, as adjusted by step 3, is indicated. There are no appreciable currents in Gaillard Cut, but currents up to 4.5 knots would obtain in the proposed canal; therefore, an appreciable increase in width is indicated. The alinement of the proposed canal would be an improvement over the existing alinement through Gaillard Cut, so a decrease in width is indicated. Bank material would be the same for both waterways, so no change in width is indicated because of this factor. Gaillard Cut is about 8 miles long, and the proposed canal would have about 30 miles of restricted channel of minimum width; therefore, an appreciable increase in channel width is indicated. These indicated adjustments, which apply to both case 1 and case 2, Fig. 57, for the Gaillard Cut, and the indicated adjustments as developed for the Cape Cod Canal, the Houston Ship Channel, and the Suez Canal, are shown in Fig. 57 for
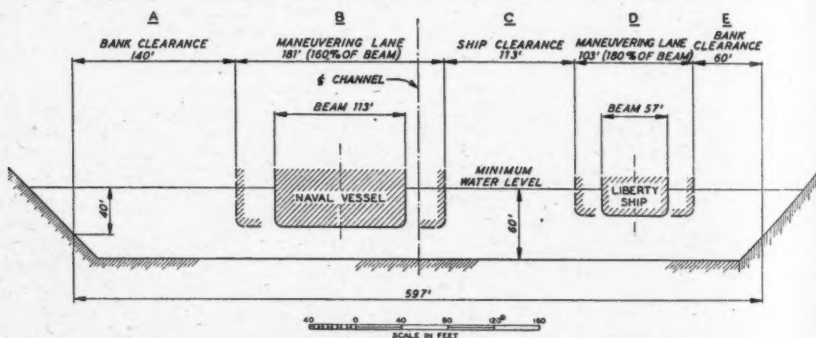

FIG. 58.—ELEMENTS OF CHANNEL DESIGN; SEA-LEVEL CANAL

both lock and sea-level canals at Panama. The characteristics are listed in the order of their importance to channel width. The general indicated adjustment for each of the four waterways and the bottom widths of each after adjustment by the first three steps are shown in Fig. 57, slight adjustments being indicated in parentheses and appreciable ones being underlined. The bottom width for the Panama sea-level canal as indicated by this method is 560 ft. However, because of variable slope conditions that would prevail throughout the canal, it is considered desirable to reference the channel width to a depth corresponding to the draft of the larger vessels, which is approximately 40 ft. On the basis of 1-on-1 side slopes used in this method, the indicated desirable width at the 40-ft depth is 600 ft. The indicated width at the 40-ft depth for an improved Panama lock canal is 500 ft.

Method B.—In this method the channel is divided into five parts (Fig. 58) and each part is analyzed separately. The combined widths of the parts then determine the width of the channel. The parts are titled "bank clearance"

(A and E), "maneuvering lanes" (B and D), and "ship clearance" (C). In this method of analysis, use is made of experience at Panama, pilot opinion, and the results of the ship model tests as presented in the fifth paper of this Symposium. The passing of the large naval vessel and the Liberty ship is used in this application since test data are not available for the ore ship.

The bank clearances A and E are considered as the space from the rail of the ship to the bank at a depth corresponding to the draft of the vessel in question. This space must provide sufficient water area between ship and bank so that the ship will retain good controllability when transiting at this distance off the bank. This distance is selected from Fig. 59, which gives the rudder angles required to hold a ship on a course parallel to the bank for various offsets from the bank of a channel 60 ft deep and 600 ft wide at the 40-ft depth. The angularity of the rudder that had to be carried to hold or to maintain the vessel on its course under a particular set of conditions is considered the best measure of controllability. The maximum angle to which a ship's rudder can be set is normally about 35°. T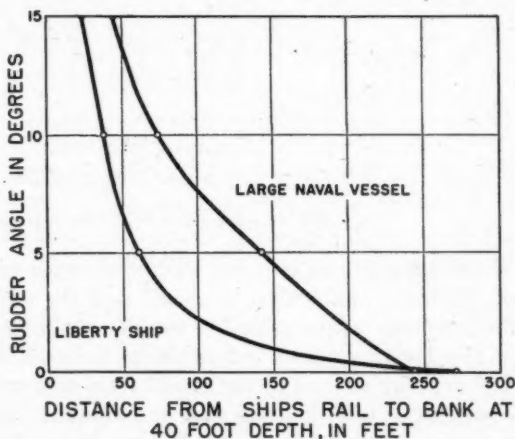he difference between this angle and that required to accomplish a particular maneuver is a measure of the reserve controllability

FIG. 59.—BANK CLEARANCE VERSUS RUDDER ANGLE

available in the event of an emergency. Thus, a large rudder angle indicates a low safety factor, and a small angle, a high safety factor. The selection of the maximum rudder angle considered safe for approaching the bank or for passing an opposing ship must, therefore, be made primarily on the basis of navigational safety. Also, the selection must take into consideration the discrepancies in model tests as compared to actual navigation, and the factors affecting the channel dimensions which are not fully considered in operating the model. The bank clearance selected is based on a rudder angle of 5°, considered to be a very conservative value but selected to provide an excess of safety to cover conditions that exist in an actual waterway that cannot be reproduced in the model. From Fig. 59, the bank clearance for the large naval vessel is 140 ft, and for the Liberty type ship it is 60 ft—both clearances being measured at the 40-ft depth. The curves in Fig. 59 are based on a 9-knot speed rather than on a 6-knot speed—to take account of head currents.

The maneuvering lanes are indicated as B and D, Figs. 58 and 60. These lanes represent the part of the channel in which each ship may maneuver without encroaching on the safe bank clearance or without approaching the other ship so closely that dangerous interference between ships will occur. The ship

model tests in themselves did not give sufficient data for fixing the widths of the maneuvering lanes because time and funds did not permit the running of a sufficient number of tests on the wide range, of ship sizes and types that would be necessary to establish the possible variations of a ship's position when meeting. Based on pilots' opinions and observation of ship courses, the width
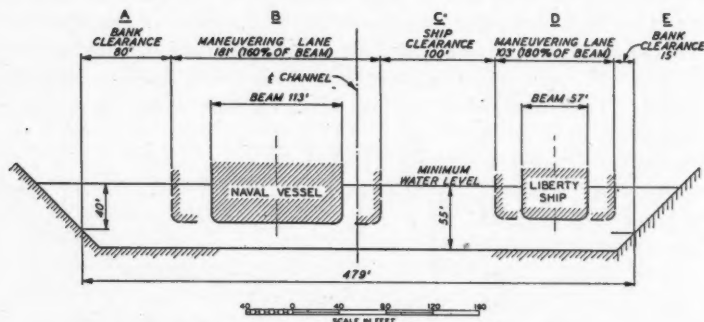

Fig. 60.—Elements of Channel Design; Lock Canal

of the maneuvering lane has been established as ranging from 160% to 200% of the beam of the ship, depending on the characteristics of the ship. The limited observations taken on transits of actual ships through certain reaches of the Panama Canal indicate that these allowances are entirely adequate. Plots of ship courses in the model tests confirm the allowances. Neither experience nor tests indicate that currents have an appreciable effect on the required maneuvering lane if sufficient depth of channel is provided to assure proper response to the rudder. Ships having "very good," "good," and "poor" controllability are considered to require maneuvering lanes of 160%, 180%, and 200% of their beams, respectively. The large naval ship with a beam of 113 ft, therefore, would require a lane width of 181 ft; and the Liberty ship, one of 103 ft.

   The ship clearance indicated as lane C in Fig. 58 is the minimum space between two ships in passing that will assure good controllability for both ships during and following the passing maneuver. As both ships approach the inner limits of their maneuvering lanes, the effects of the channel banks decrease, and the interaction between the two ships becomes controlling. With ships at the inner limit of the maneuvering lane, bank action is insignificant. The ship model tests indicate that, if rudder angles of from 10° to 15° are not to be exceeded when two such ships pass in a channel with a current up to 4.5 knots, the distance between ships must be at least 100 ft or equal to the beam of the larger ship, whichever value is greater. Pilot opinion is consistent with this criterion in establishing ship clearance. A larger rudder angle for passing than that used for bank clearance is justified because of the much shorter time that the interaction between ships exists as compared to the time bank action may affect the ship. The minimum clearance required for the passing of the large naval vessel and the Liberty ship is 113 ft.

With a computation based on this method of analysis, the required width of the sea-level canal, measured at a depth of 40 ft, would be 597 ft. The values for each part of the channel are shown in Fig. 58. A similar analysis for the minimum section of a lock canal resulted in the values shown in Fig. 60.

The Panama Canal pilots assisting in the ship model tests were of the opinion that the effect of the proximity of channel banks and channel bottom, the currents, and the interaction between passing ships were clearly and accurately indicated in the ship model tests. A limited series of prototype tests at Panama has generally confirmed qualitative results of the model tests. However, there are certain discrepancies between model and prototype which must be considered in adapting the test data to channel design. The factors favoring the test results included the following:

   (*a*) Uniformity of channel section,

   (*b*) Weather conditions; and

   (*c*) Freedom from mechanical (rudder and engine) breakdowns.

Those factors not favoring the test results included the following:

   (*a*) Time scale requiring pilots to act seven to ten times as fast;

   (*b*) Stability of ship models about a vertical axis not being comparable with prototype ships (particularly evident in the test run using the 1:86 scale ratio); and

   (*c*) The fact that the pilot is not actually aboard the vessel and thus loses the viewpoint and the "feel" of an actual ship.

Pilots were in agreement that navigation under prototype conditions would show some over-all improvement over the action observed in the model. The model tests, although limited in their capabilities to reproduce and take into consideration all factors affecting the safety and security of transiting ships in an actual waterway, have performed an invaluable service in enabling the pilots, technicians, engineers, and consultants to reach a common understanding of the operation of ships in restricted channels.

FINAL SELECTION OF CHANNEL DIMENSIONS

Pilots and marine operating officials at the Cape Cod Canal, the Houston Ship Channel, and the Suez Canal were polled informally by Panama Canal representatives on their views with respect to the required channel width and depth for two-way traffic in the Panama sea-level canal. The Cape Cod personnel considered that a channel from 500 ft to 600 ft wide, and of adequate depth, would afford easy navigation for the largest commercial and military ships. At the Houston Ship Channel it was considered that for a channel with rocky banks the channel should have a minimum width of 600 ft and a minimum depth of 60 ft. The Suez Canal personnel were of the opinion that a channel from 500 ft to 600 ft wide and from 50 ft to 60 ft deep would be ample for two-direction traffic of the largest commercial vessels operating in currents up to 4.5 knots.

The Marine Division of The Panama Canal and its pilots have been closely associated with all navigation studies made as a part of "Isthmian Canal

Studies—1947," and they are in general agreement that a channel 60 ft deep and 600 ft wide at the 40-ft depth would be satisfactory for the sea-level canal and that 55-ft by 500-ft dimensions would be adequate for the restricted channels of the lock canal.

As a result of the design studies and opinions of marine operating personnel, it was concluded that the Panama sea-level canal should have a minimum channel depth of 60 ft and a minimum channel width of 600 ft at the 40-ft depth, and that corresponding dimensions for an improved Panama lock canal should be 55 ft and 500 ft.

CURVES

Navigation around curves is generally more difficult than in a straight channel, particularly if currents exist. The alinement as tentatively accepted for a sea-level canal has a maximum deflection angle of 26°. This value corresponds to maximum deflection angles of 30° for the Gaillard Cut of the present Panama Canal, 63° for the Suez Canal, 75° for the Cape Cod Canal, and 109° for the Houston Ship Channel.

Experience in Existing Waterways.—Analysis of the different types of curves on existing waterways, although illustrating the weaknesses of much of the treatment, has not resulted in the development of a satisfactory method of treatment for a sea-level canal. There are so many differences in operating technique and physical conditions of the different waterways that comparison of curve treatment to establish the best method is not possible. The marine opinion at Suez, where the original curves were widened and modified, is that unwidened long-radius curves are the most desirable. However, this opinion is based on single-direction traffic on curves with no passing even of ships tied up to the bank.

The Cape Cod Canal is a good example of a channel having curves of adequate radius (7,000 ft or more) and channel width (480 ft) to permit two-direction traffic and practically unrestricted operation by all ships that may transit the waterway. There are no straight sections in the land-cut channel of the Cape Cod Canal of sufficient length to establish any relationship between the difficulties of navigating on a curve as compared to a straight channel.

At the existing Panama Canal a true curve is not used. In most cases the outside prism lines are extended to intersection. The inside prism lines are stopped short of the point of intersection and connected by a chord. This treatment, although reasonably satisfactory for a channel without currents, would be unsatisfactory with currents because of poor flow and eddy conditions.

Ship Model Tests of Curves.—Ship model tests of various types of curve treatment were performed at the Taylor Model Basin. Time and funds did not permit the continuation of these tests to the point where finally conclusive results could be established. Most of the tests were conducted at a scale of 1:86 rather than at the scale of 1:45 used in straight-channel tests. Outlines of the various curve treatments tested are shown in Fig. 52. The 1:86 scale tests indicated that the curve with no widening (Fig. 52 (*a*)) was superior to all the others for single-direction traffic and that two-direction traffic would not be acceptable for any of the curves. The results of the 1:86 scale tests were very

erratic and definitely not consistent with actual experience at either Cape Cod or Suez. It was believed that the instabilities of the 1:86 scale model and of the 9:1 time ratio were the principal causes of the unsatisfactory results. Therefore, tests were made using a scale ratio of 1:45, of a single unwidened curve, 600 ft wide at a 40-ft depth and 60 ft deep, with 1-on-1 side slopes, a 12,500-ft radius, and a 26° intersection angle. These tests clearly demonstrated that complete dependence could not be placed on the results of the 1:86 model tests. The tests to a 1:45 scale demonstrated that single-direction navigation around a prototype of the curve tested is entirely practicable and safe with controllability characteristics similar to those for operation in straight channels. Time did not permit two-way navigation tests in the 1:45 scale curve. The limited number of off-center runs, although showing some improvement in conditions over the 1:86 scale tests, did not indicate that two-direction traffic of "design ships" would be a safe operation.

Whether a practicable method of widening can be developed to permit passing of the large ore carrier or naval vessel and a loaded Liberty ship (design criteria for straight channel) has not been established. It is considered that model tests to scales of 1:20 or of 1:30 should be used in any further tests to determine the best methods of curve treatment.

Should it develop that satisfactory curve treatment for two-direction traffic is unattainable, operating restrictions would have to be placed on the passing of the larger ships on curves. These restrictions would not noticeably affect the transit time or increase delays to shipping but would necessitate more detailed dispatching.

ALINEMENT

The alinement of the present canal contains twenty-three angles with a total angularity of 598°. The largest of the angles is 67°, and four of the angles are larger than 50°. Use of this alinement and a center-line radius of 12,500

TABLE 29.—COMPARISON OF SELECTED ALINEMENT FOR SEA-LEVEL CANAL WITH PRESENT CANAL ALINEMENT

Canal	No. of Angles		Angularity (Degrees)			Length (Miles)		Restricted Channel	
	<20°	>20°	Maximum angle	Total	Per mile	Minimum length of straight reach	Total length of canal	Length (miles)	In-curves (%)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
Sea level......	6	2	26	117	2.54	2.94	46.02	30	14
Present lock...	11	12	67	598	11.68	0.07	51.20	12	33

ft on the curves would result in almost 50% of the restricted channel of a sea-level canal being in curves, and was considered unsatisfactory for a sea-level canal having a restricted channel for the major part of its length in which currents up to 4.5 knots might exist.

An investigation was made of possible alinements for a sea-level canal at Panama ranging from the present canal alinement to an absolutely straight

alinement from shore line to shore line.   The proposed alinement is shown in
Fig. 7.   The maximum angle in this alinement is 26° and the total angularity
is 117°.   It was found that any reasonable increase in the size of the maximum
angle or in the total angularity would not decrease the excavation cost.   A
comparison of the characteristics of this alinement with that of the present canal
is shown in Table 29.

Summary of Sea-Level Channel Design

A summary of the tentative channel design of a sea-level channel resulting
from these studies and investigation is as follows:

Physical Dimensions—
    Total length of restricted section of minimum channel width,
      in miles... 30
    Minimum Channel Section—
      Depth, in feet...................................... 60
      Width, in feet, at 40-ft depth...................... 600
      Side slopes (steepest)............................. 3 on 2
    Number of angles in restricted section.................... 6
    Maximum angle, in degrees............................. 26
    Radius of curves, in feet............................... 12,500
    Widening at curves (dependent upon further study)........ ...
    Minimum distance between points of intersection, in miles.... 4.21
    Minimum sight distance, in miles........................ 1.52
Qualifying Operation Criteria—
    Pilots required for all ships.............................. ...
    Maximum transit speed (water speed), in knots............ 10
    Normal passing speed of large ships, in knots.............. 6
    Two-direction traffic with maximum limitation of largest ex-
      isting naval vessels or a large commercial cargo vessel pass-
      ing an average size commercial cargo vessel.............. ...
    Passing on curves limited to average size ships (subject to ad-
      ditional curve study)................................ ...
    Maximum length of ships, in feet, that can pass in same direc-
      tion... 300

The design is liberal in physical dimensions and should provide a safe and
efficient waterway for all shipping, both commercial and military, that may be
expected to use it to the year 2000.

The adequacy of the proposed sea-level canal channel is affirmed by Panama
Canal pilots as presented in a review of the Governor's Report to Congress by
a pilot committee consisting of four senior pilots whose total experience at the
Panama Canal totals 87 years:

> "From a piloting point of view, we feel that this canal [sea level] will
> have a greater safety factor than the other three plans [lock canal] outlined
> and will not contain any difficulties or hazards of any importance.   The
> most important safety factors inherent in the proposed sea-level canal are

the dimensions, alignment, and tidal-control structures, as shown and described in the draft [of the report]. We are of the opinion that in the event the tidal-control structure should be badly damaged or destroyed, it would be feasible to operate the canal as an uncontrolled waterway."

Further Tests and Investigations.—The channel dimensions for the Panama sea-level canal, as developed in this paper, are considered to be satisfactory. However, in the event that construction is authorized, the magnitude of the project and the necessity for more information on curve treatment would make it essential that additional studies be made. These studies would consist of further investigations of navigation in existing waterways and continuance of the ship model tests. Particular emphasis would be placed on the development of data to confirm the width of maneuvering lane and passing clearance and the best method of treating curves. Model studies would be conducted at a larger scale than used previously to reduce the scale effects apparent in former studies.

EXCAVATION SLOPES

BY WILSON V. BINGER,[43] ASSOC. M. ASCE AND THOMAS F, THOMPSON,[44] AFFILIATE, ASCE

SYNOPSIS

The geologic and soil mechanic studies conducted to establish stable slopes for the formations that would be excavated for a sea-level canal at Panama are described in this paper. The authors discuss the early and current geological investigations, the geological features of the area, the difficulties experienced in the slides that occurred during construction of the present canal, and the development of slope standards used in the preliminary design of a sea-level canal.

The paper concludes that adequate exploration and testing to develop the strength properties and distribution of the materials to be excavated, combined with present understanding of the mechanics of slide development, allow design of slopes that would be secure from major slides.

INTRODUCTION

The construction of a sea-level canal across the Isthmus of Panama would require the excavation of more than 1,000,000,000 cu yd of material, or about three times the volume of material removed in the construction of the existing canal. Almost 30% of the new excavation would be concentrated in a 4-mile section through the Continental Divide north of Pedro Miguel, where cuts deeper than 600 ft would be required.

Experience with the great slides that developed during the excavation of the present canal indicates the design of stable slopes for a sea-level canal to be a problem of unprecedented magnitude in engineering geology and soil mechanics. The renowned slides of the Panama Canal occurred in the Continental Divide area, the greatest troublemakers being limited to the 1-mile length of deepest cut where the weak Cucaracha formation in the East Culebra, West Culebra, and Cucaracha slides (Fig. 61) increased by about 20% the amount of excavation required for building the canal. These slides are object lessons for the design of slopes.

There can be little question of the importance of designing the canal banks for stability. If permitted to occur, slides might block the canal during or following construction. Interruption of traffic by slides at any time would penalize commerce, and in wartime such delays would be intolerable. Furthermore, if slides occur, the total amount of material to be removed would be greater than the excavation required if the canal banks were excavated initially to stable slopes.

FIG. 61.—LOCATION OF MAJOR SLIDE AREAS, PANAMA CANAL.

The current studies for Public Law No. 280 have been greatly facilitated by the extensive slope studies made from 1939 to 1942 during the Third Locks Project, particularly those of the areas containing the materials most critical with respect to sliding. Despite the extent of the past and current studies, additional geologic exploration would be required prior to excavation to locate the contacts between rock formations more accurately, as well as to define the location and character of the various weak and strong strata within each formation. More extensive testing and more detailed analysis would also be employed in the design of final excavation slopes. It is believed, however, that the studies of slopes reported herein have been conservative enough that the estimates of excavation cost will more than cover the adjustments in slope design that may result from more extensive studies.

History of Geologic Exploration

The geology of the Canal Zone has been studied intermittently for many years, but such study prior to the arrival of the French was not conducted for engineering purposes and, moreover, was neither systematic nor thorough. By present standards, the geologic investigations by the French during the periods of the first and second canal companies were very inadequate. Explorations were conducted in some detail to determine foundation conditions at dam, lock, and harbor-facility sites by both the early French and early American investigators prior to commencing construction, but only a minimum of information for establishing proper slopes was assembled for the regions of heavy cut through the Continental Divide area where disastrous slides later developed.

TABLE 30.—Subsurface Explorations, Panama Canal, 1881 to 1947

Line	Agency	Years	Total number of holes	Holes in Gaillard Cut	
				No.	Average depth (ft)
(1)	(2)	(3)	(4)	(5)	(6)
1	Old French Company.............	1881–1894	605	63	{ (42 holes) 120
2	New French Company............	1894–1903	60	{ 27 pits { 2 borings	No records available
3	Isthmian Canal Commission........	1904–1914	5,073	81	160
4	Third Locks Project[a].............	1938–1943	1,968	242	151
5	Studies under Public Law No. 280[a]..	1946–1947	230[b]	87[c]	335

[a] The Panama Canal.   [b] Includes twenty-three holes on adjacent Panama routes.   [c] For purposes of estimating sea-level canal slopes, these holes are of more value than all previous borings combined. They are the only borings completed for that specific purpose—that is, spotted at critical locations, carried down to sufficient depth (El. −100 or deeper), with cores carefully logged by a geologist, and with samples of weakest materials tested in the laboratory.

The principles of slope design to forestall the "deep deformation," shear-type slides were not known at the time of the early studies. It was not until near the end of the American construction period, after serious sliding was well advanced, that the studies of the late D. F. MacDonald, then resident geologist, led to an appreciation of causes and prevention of this type of failure.

Studies for the design of the Third Locks Project between 1939 and 1942 resulted in a thorough understanding of the geologic nature of the relatively

small areas in which excavation was to be made for the locks and approach channels. In addition to determining the foundation adequacy of the rock at the sites selected for locks and appurtenant structures, investigations were sufficiently detailed to permit the establishment of slopes that would be secure against major slide development. In this work great benefit was derived from the services of Mr. MacDonald, who had returned to the Canal Zone as consulting geologist.

Table 30 presents the quantities of subsurface exploration performed during each of the several periods of major design or construction at Panama.

FIELD AND LABORATORY INVESTIGATIONS

The investigations of the geology of the region traversed by the alinement of the Panama sea-level canal included: (1) Mapping on 1:20,000 scale base maps of all outcrops to be found along the line of the present canal, roads, and natural exposures such as stream channels that have removed the deep soil overburden; and (2) core borings at selected locations to develop further the character, distribution, and structural relationships of the underlying rock formations. Both methods of investigation benefited from stereoscopic examination of aerial photographs, which permitted characteristic topographic expressions to be used in tracing faults and contacts and in establishing the areal extent of certain formations. A large proportion of the borings was located near the Continental Divide where the sliding of oversteep slopes in the original canal excavation attested to the need for careful study in setting of slopes required by the greater depths of cut for the new canal.

The deepest boring made during the present investigations, near the Continental Divide, was 825 ft deep. The average depth of all holes drilled was 236 ft, and the total footage drilled for the 230 holes completed up to July 1, 1947, was 54,376 ft. Drill accessories employed were of "NX" size and yielded 2$\frac{1}{4}$-in. diameter cores. Tungsten-carbide hand-set bits were found to be most efficient for the soft and medium rock types, whereas diamond bits proved best for the hardest igneous rocks.

Core-boring operations were inspected by a qualified geologist assigned to each drill rig. Samples of rock were obtained with standard core barrels and samples of the softer types of plastic overburden with 3-in.-diameter brass tubes. Samples were subjected to standard laboratory tests to determine their engineering properties, of which the strength characteristics were the most important and were determined principally by unconfined and triaxial compression tests.

GEOLOGY

Geologic History and Physiography.—The datable geologic history of the Canal Zone commences in late Eocene time, some 50,000,000 years ago, when the local Isthmian region was depressed to the extent that a seaway extended diagonally across it and covered at least part of the present Canal Zone. During the succeeding ages, the land surface was repeatedly raised and lowered by diastrophism, resulting in changes in base level that are reflected in the character and distribution of bedded formations deposited on the lands or in the seas of each period. Recurrent volcanic activity ejected vast quantities of
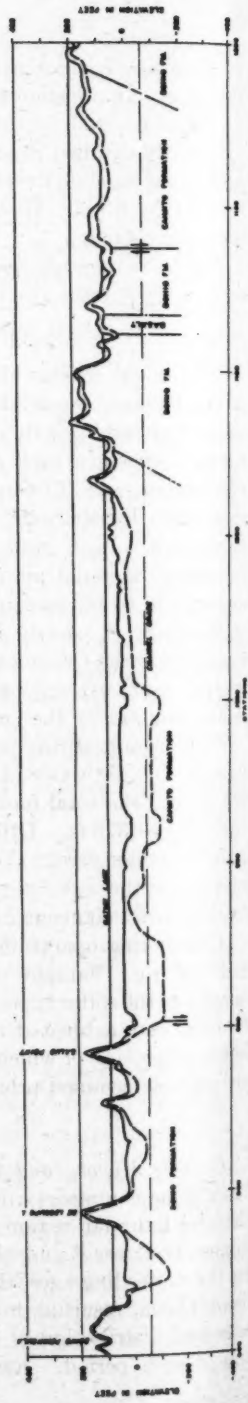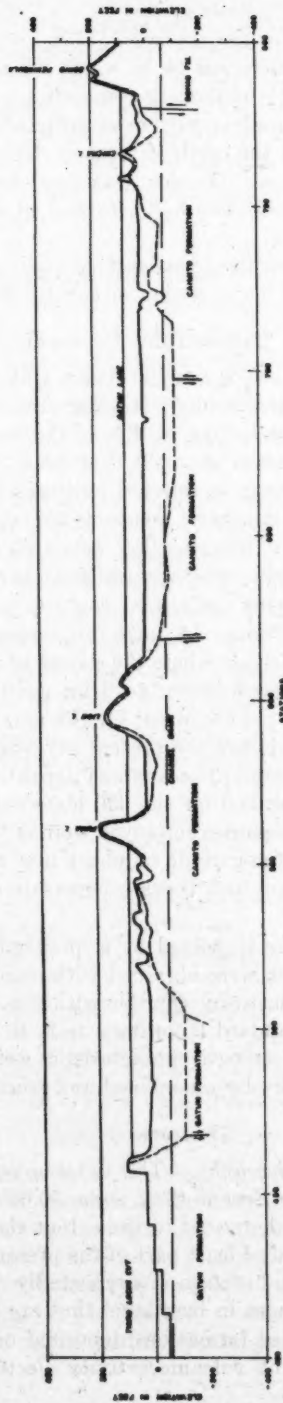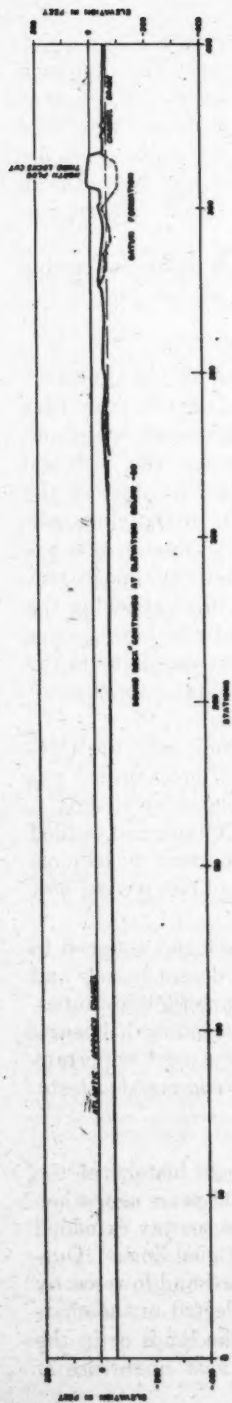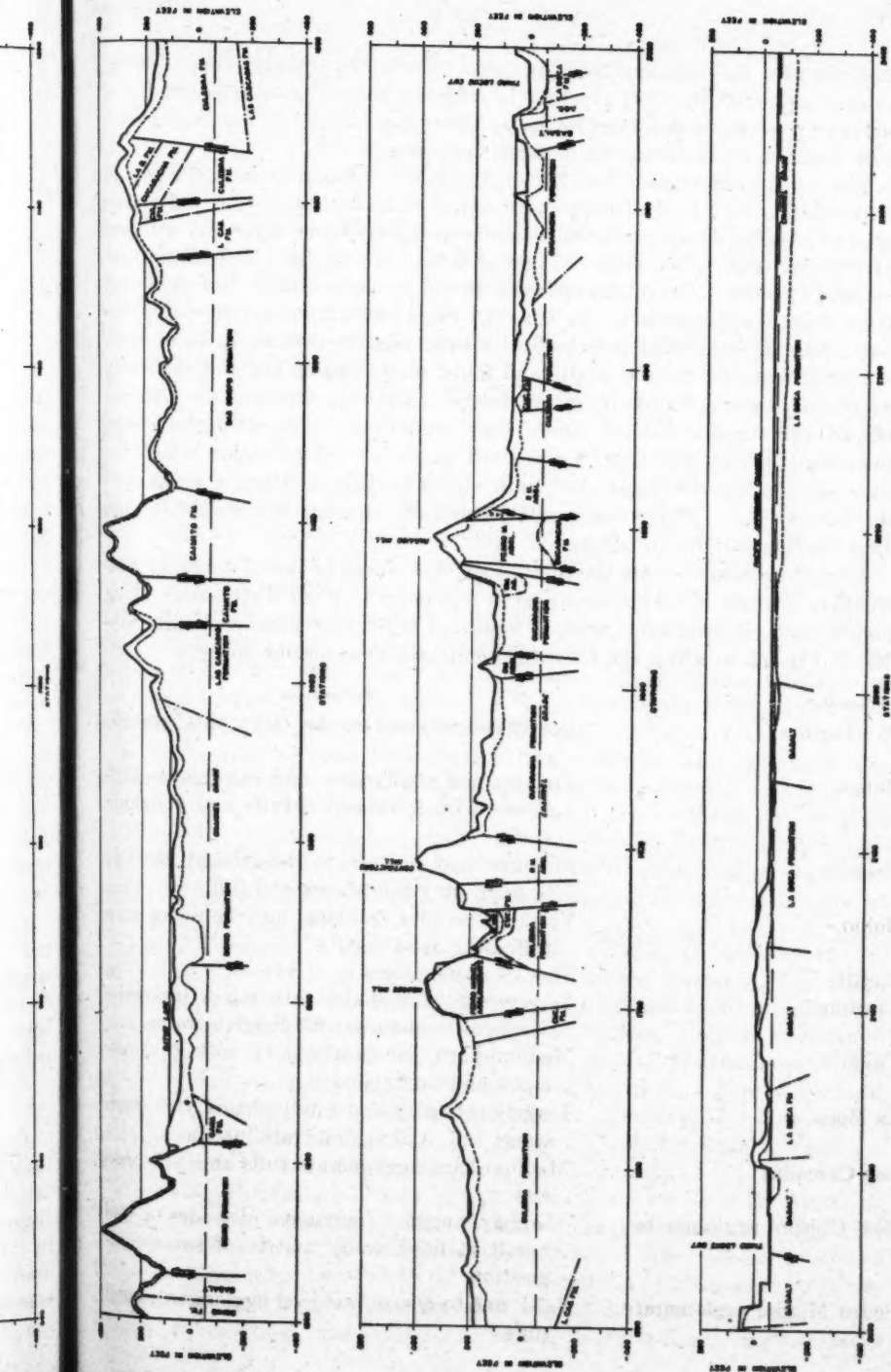
Fig. 62.—Geologic Section Along Center Line, Panama Sea-Level Conversion Route

materials over then-existing lands and seas or intruded igneous rocks into the pre-existing materials. This period is reflected today by widespread distribution of pyroclastic flow, and intrusive rock types intimately associated with those resulting from normal sedimentation processes.

The topography of the Canal Zone is the result of stream erosion in a humid climate acting on a land surface, composed of rocks with very different resistances to erosion, which periodically has been elevated and depressed with respect to sea level. The region is one of unique topographic diversity, characterized by conical, irregularly spaced hills and by unsystematic drainages that give a chaotic appearance to the terrain. Sizes and shapes developed by the land forms are controlled principally by their relative resistances to erosion. Structural features, such as faults and folds, play a secondary and relatively minor role in configuration of the landscape. Drainage patterns are well developed and sharply defined despite their comparatively recent geologic age. Areas underlain by soft rocks are marked by broad valleys within which the larger streams have beveled the strata and deposited on them a blanket of alluvial material. Along both coasts, extensive swamps are present in the lower reaches of the main streams.

Rock Formations.—Only the formations that would be cut through by construction of a new canal are described in this paper. Their distribution along the proposed sea-level canal route is indicated in the center line geological profiles, in Fig. 62, to which the following condensed descriptions apply:

Formation	Explanation
Overburden	Poorly consolidated mucks, clays, silts, gravels, etc.
Gatun	Fine-grained argillaceous and calcareous sandstones with interbedded tuffs and conglomerates
Caimito	Medium-hard, medium to fine-grained, coarsely bedded, limy sandstones and tuff
Bohio	Volcanic pebbles, cobbles, and boulders in a tuffaceous sand matrix
Basalt	Flows and intrusives
Cucaracha	Largely soft clay shales with minor interealations of sandstones and conglomerates
Culebra	Medium-hard sandstones and soft carbonaceous and sandy shales
La Boca	Largely soft silty and sandy shales with sandstone, tuff, and agglomerate interbeds
Las Cascadas	Medium-hard agglomerate tuffs and tuff-breccias
Bas Obispo agglomerate	Largely angular fragments of andesite and basalt in hard sandy matrix of same composition
Pedro Miguel agglomerate	Hard, fine to coarse textured agglomerates and tuffs

In succeeding paragraphs each formation is taken in order of geologic age, beginning with the oldest and progressing to the youngest materials.

Bas Obispo Formation.—The Bas Obispo formation is a massively jointed agglomerate formed from angular rock fragments and ash that was blown from an old volcanic vent and cemented into a strong dark-gray rock of concrete-like appearance on freshly exposed surfaces.   The Bas Obispo is hard and tenacious except where broken or sheared and softened by faulting.   Only small slides have developed in the section exposed along the existing canal, and these are related to fault-broken zones.   Where undisturbed by faulting, the formation represents one of the strongest rock types involved in new canal construction. About 2.5% of the total excavation for a sea-level canal at Panama would be through this strong formation.

Las Cascadas Formation.—The Las Cascadas formation originated in much the same manner as the Bas Obispo and directly overlies it, but its subsequent history has been dissimilar and it is much more heterogeneous.   The Las Cascadas is mostly composed of scattered hard angular fragments of basalt and andesite that are embedded unsystematically in a fine, soft to medium-hard, altered tuff.   Some sections are composed entirely of well-bedded tuffs without the hard inclusions.   The fines are variably clayey and locally somewhat bentonitic.   The formation contains interbedded and random sheets and dikes of fine-grained, hard, igneous rock, mostly either basalt or andesite.   During and after the original construction of the canal, numerous small bank failures developed in this formation where the banks had been weakened by faulting. One of the weaker rock units within the Canal Zone series, it would comprise only 1.5% of the excavation for a sea-level canal.

Bohio Formation.—The Bohio formation is composed of heavy beds of conglomerate and sandstone with infrequent shale or tuff layers.   The conglomerates are most prevalent and consist of subangular to well-rounded pebbles, cobbles, and boulders in a basaltic sand matrix.   The Bohio, in general, is well-cemented, moderately hard, and has widely spaced joints.   This formation would comprise roughly 12.5% of the total excavation for a sea-level canal at Panama.

Culebra Formation.—The Culebra formation overlies, unconformably, the Las Cascadas formation and is in turn overlain by the Cucaracha formation. The Culebra can be divided into an upper calcareous and a lower argillaceous member.   Its component beds are mainly soft to medium-hard sandstones, shales, limestones, tuffs, and thin conglomerate seams.   Carbonaceous shales and a few impure lignite beds are found frequently in the upper and middle parts, and it is highly fossiliferous throughout.   This is one of the weaker formations found in the Canal Zone.   About 15% of the sea-level canal excavation would be rock of this formation.

Cucaracha Formation.—Strata of this formation were involved largely in the historic huge Gaillard Cut slides that developed during and continued for some time after the original digging of the canal.   The maximum known thickness of the formation is 625 ft.   Its composition is dominated by weak, poorly bedded, variably bentonitic, slickensided, soapy-textured clay shales (altered impure tuffs) interbedded with soft to medium-hard, fine, tuffaceous

siltstones; medium to coarse, cross-bedded sandstones; pebble conglomerates; thinly bedded, often lenticular, soft clayey lignitic beds; and one hard bed of agglomeratic, indurated tuff. The formation largely represents an accumulation of fine volcanic detritus that has been reworked by stream action and subjected to a partial chemical decomposition of its component ash particles with resulting creation of hydrous clay minerals of the montmorillonite-beidellite group. It is the weakest rock formation encountered along the Panama sea-level canal alinement and would represent 14.5% of the total volume of excavation for sea-level construction.

La Boca Formation.—The La Boca formation consists of medium-hard, silty or sandy, variably calcareous shales, tuffs, sandstones, and limy concretion beds, with scattered heavy agglomerate layers. It is similar in most essential respects to the Culebra formation but represents only 2% of the material to be excavated for sea-level canal construction.

Pedro Miguel Agglomerate.—This formation occurs near Pedro Miguel, where it overlies and is in intimate association with the Cucaracha formation. Thick beds of agglomerate also are found interspersed within the lower part of of the La Boca formation. The dominant constituents range from small angular fragments to huge blocks of basalt enclosed in a strong, dense tuff matrix, locally cemented by secondary calcite. Beds of hard, black, indurated tuff are found through its section. It is massively jointed and frequently shows crude bedding. It is one of the stronger types of material to be found locally. Only 4% of the materials that would be removed for a sea-level canal are from this formation.

Caimito Formation.—The Caimito formation is widely distributed in the Gatun Lake area, where it overlies the Bohio formation. It has been divided into three recognizable units. Its basal phase is a tuffaceous sandstone conglomerate of spotty, localized outcrop distribution containing abundant igneous pebbles, cobbles, and boulders. The middle phase, slightly fossiliferous tuffs, limestones, and sandstones, is also localized in extent. The upper phase is a widely distributed series of tuffs, tuff-breccias, and sandstones, with occasional sandy or fairly pure limestone beds. It is moderately strong throughout. Of the total sea-level canal excavation, 13.5% would be through its members.

Gatun Formation.—This formation is composed essentially of variably calcareous or argillaceous fine-grained sandstones, tuffs, and a few conglomerate beds that were deposited in a shallow sea in middle Miocene time. Stratification and jointing are massive and tight. As encountered at the site for the Gatun Third Locks, it was easily excavated with a minimum of blasting, and the deep cuts made for this project in 1941 and 1942 show no sliding after 5 years. Rock of this formation would comprise 2% of the material to be removed in sea-level canal construction at Panama.

Atlantic and Pacific Mucks.—These deposits are the combined accumulation of stream-deposited and ocean-deposited fines that were laid down as a result of a general land submergence in late Pleistocene time that drowned the then-existing stream-carved topography. Soft clays, silts, sands, organic swamp deposits (mostly decayed leaves and wood), and layers of sea shells are found

intimately intermixed and to highly variable depths. Near Gatun Dam, muck-buried channels locally extend to below El. − 200. The mucks have very high natural water contents and are of low shear strength. They can be removed economically by hydraulic dredging methods.

Igneous, Intrusive, and Flow Rocks.—All formations of the Canal Zone older than the Gatun (Middle Miocene) have been intruded by volcanic dikes, sills, or plugs, and have interlayered flow rocks. These are mostly basalt, but andesites, rhyolites, and dacites are occasionally present. Rocks of this group in their unweathered occurrences are hard crystalline types and are characterized by a high shear resistance, but their other properties are extremely variable. They are found in greatest development in the central and Pacific regions. About 8.5% of the total volume of sea-level excavation would consist of igneous rocks.

Soils and Weathered Rock.—Depth and character of the soil and weathered rock cover are much diversified and are usually related to the type of underlying rock and topography. In many instances, weathering is known to extend to depths of 50 ft or more, but the average depth is in the order of from 20 ft to 30 ft. Clay soils predominate throughout but are usually rather stiff and only moderately plastic. This material and the mucks previously described represent together 24.5% of the total volume of materials requiring removal for new canal construction.

Geologic Structure.—Geologic features cut by the center line of the proposed new canal alinement are shown in profile in Fig. 62. Particular geologic study has been devoted to a number of basalt-capped or agglomerate-capped hills in the section between the Continental Divide and Pedro Miguel. The structural relationships of these hill-forming, hard basalts and agglomerates to adjacent or underlying softer materials of relatively low shear resistance, as typified by the weak Cucaracha beds, are of great importance. Certain hills have been determined to be deep-seated, stable masses of agglomerate or basalt that present no unusual problem in the design of excavation slopes for a canal, but others involve basalt or agglomerate, as either a cap or an overhang, resting on the Cucaracha formation. Experience in the construction of the existing canal has demonstrated that severe slides may result in cases where a deep cut is opened in a competent mass underlain by weak material if the contact between the two materials is above the bottom of the cut or within limited distances below. The potential slide hazard of this condition can be eliminated only by special slope-design treatment involving a removal of at least a part, if not all, of the overlying hard rock mass above the line defining the stable slope of the weaker formation.

PANAMA CANAL SLIDES

Types of Slides.—The slides of the Panama Canal were divided by Mr. MacDonald into four types. His description of these follows:

"(1) Deep-deformation Slides. These began where the canal channel was cut deep into weak, somewhat plastic rocks. The weight of the high steep banks caused the rocks to deform very slowly and eventually to shear more than a score of yards below the bottom of the excavation. (2) Structural Slides. These took place where sheared zones, joints, bedding planes, or

other structural weaknesses dipped fairly steeply toward the excavation. (3) Mudflow Slides. These formed where mud or moist clay slipped off or flowed down an inclined surface. (4) Combination Slides. A combination of any of the above types."

The deep-deformation slides were by far the largest and most troublesome. They practically all originated in the Cucaracha formation and, in the area of principal occurrence, are represented by the two slides of greatest magnitude, the East and West Culebra. Slides of this kind began to develop when the cut reached a depth of about 100 ft and increased in size and frequency as the excavation was deepened. The next largest slide was the Cucaracha slide, of the mudflow type. The structural and combination slides were not nearly as large or as troublesome as those of the deep-deformation type. The brief slide history which follows is confined to the East and West Culebra slides and the Cucaracha slide, because they were not only the largest, but also the only slides that actually blocked the channel. The locations of these three slides are shown in Fig. 61.

Slide History.—Shortly after the start of excavation by the first French canal company in 1884, the Cucaracha slide became active. It appeared at first to involve only the surface materials—soil and highly weathered rock from 20 ft to 30 ft thick—and took the form of mudflows down the steep slopes of the cut. This slide was intermittently active between 1885 and 1889, while the first French company was in operation. From 1889, when this company failed, until 1905, the slide was small or inactive, there being practically no excavation in the Cucaracha area by the second French company.

The first French company attempted to reduce the water in the slide material by surface drainage and tunneling. Surface drainage was reasonably successful in diverting the water, but neither method was successful in preventing the reoccurrence of slides.

American excavation for the Panama Canal began in 1905. The Board of Consulting Engineers established canal slopes approximating 3 vertical on 2 horizontal for all cuts in rock through the region of the Continental Divide.

The Cucaracha slide resumed activity in January, 1907, and in October of that year a large mass flowed into the bottom of the cut and caused extensive damage. The slide involved some 500,000 cu yd of material and for 2 weeks moved at the rate of 14 ft per day.

Another slide developed in January, 1907, on the east bank of the canal opposite the village of Culebra, and in October a crack developed 50 ft or more back of the top of the west bank near Culebra. This cracking was followed by bulging of the bottom of the cut. These movements marked the beginning of the East and West Culebra slides. The activity of the slides increased in 1911, and large quantities of material moved into the cut during the years from 1911 to 1913. During this period, excavation operations were delayed as railroad tracks and shovels were overturned. The disruptive effect of these slides is illustrated in Fig. 63.

In 1911 a project was undertaken to stabilize the banks by terracing to flatten the excavation slopes. This work was stopped in December, 1913, when it appeared that the Culebra slides were finally stable.

The Cucaracha slide started anew in January, 1913, and by February it was estimated that from 2,000,000 cu yd to 3,000,000 cu yd of clay and rock were in motion. By October, 1913, the channel was completely obstructed for 90 ft near Gold Hill, and partly blocked for 100 ft south. For the remainder of 1913 and during the first half of 1914, the slide continued to move while failed material was removed from the toe. By August 15, 1914, a channel 150 ft wide and 35 ft deep had been excavated through the Cucaracha slide, and the canal was opened to commerce.

FIG. 63.—EAST CULEBRA SLIDE SHOWING UPHEAVED MATERIAL BETWEEN STATIONS 1746 AND 1758, FACING SOUTH, FEBRUARY 6, 1913

Two months later, on October 14–15, a section of the east bank of the cut, north of Gold Hill, extending over 2,000 ft along the face of the cut and about 1,000 ft back from the center line of the channel, settled almost vertically, and about 725,000 cu yd of rock and earth were squeezed out or heaved up into the channel prism. The channel had 45 ft of water when the movement started. An hour later the bottom of the channel had been forced upward in some places to within 9 in. of the water surface. The dredges were able to clear the obstructions and keep the channel open until August 7, 1915, when slides closed the canal for 4 days. The canal was again closed to shipping for 6 days beginning September 4, 1915.

On September 15, 1915, a section of the face of Zion Hill (directly opposite the East Culebra slide) broke away and settled down. Both the East Culebra and West Culebra slides then began to move rapidly toward the channel. An

island first appeared near the middle of the canal and later formed a complete barrier, 250 ft wide at the water surface, rising 65 ft above the water level. The canal remained closed to traffic for 7 months until April 15, 1916.

The Cucaracha slide again became active on August 30, 1916, and closed the canal to traffic for 8 days. The East Culebra slide started moving again in January, 1917, and closed the canal to traffic for 2 days. No further closures of the canal were experienced until March 20, 1920, when the Cucaracha slide again broke loose to close the canal for 4 days. In 1931 the canal was closed to traffic for 2 days while a small slide in the East Culebra area was removed. This was the last time that the canal was closed because of slides.

DESIGN OF SLOPES

Cucaracha Formation.—All the worst slides of the original canal construction involved the weak, slickensided clay shale of the Cucaracha formation. Since the alinement for a new sea-level canal is through an area where the Cucaracha formation is present, the design of slopes in this material is of the greatest concern.

The Cucaracha slope-design problem was studied intensively during the Third Locks investigations, since an important part of the excavation for the new Pedro Miguel locks would have been in this material. The studies included laboratory and field tests of the undisturbed clay shales, analyses of the slides along the existing canal, and the analysis of an existing stable slope in the Cucaracha formation.

FIG. 64.—EXCAVATION SLOPE CURVE, CUCARACHA FORMATION

Difficulties were encountered in obtaining suitable laboratory samples from the initial core-drilling operations, as the cores recovered from the weaker horizons were often badly broken. The subsequent use of a heavy mud slurry in place of drill water reduced caving and increased the percentage of recovery of undisturbed samples through the more critical and weak horizons.

Many of the samples that were recovered broke apart on the slickensided surfaces when being prepared for testing. As a result of these difficulties, the weakest materials could not be tested, and the average strengths determined were regarded as high.
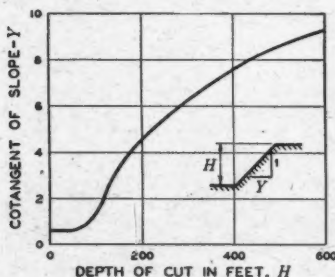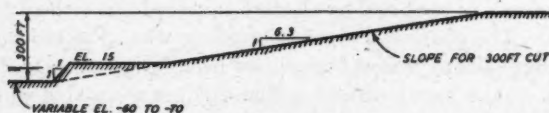

FIG. 65.—TYPICAL SLOPE BASED ON SLOPE CURVE IN FIG. 64

Since the strength of the Cucaracha formation could not be determined reliably by laboratory tests, the slope design standards for Cucaracha (Figs. 64 and 65) were based on the strength of the material derived from analytical studies of a stable bank cut in Cucaracha, combined with the results of a simple

laboratory test to measure the friction that might be expected along the slickensides in the material.

The tests to determine the friction were made in a standard direct-shear machine on specimens cut from solid cores of sound clay shale, polished to simulate slickensides, and brought into contact under water. The minimum angle of friction determined by three series of such tests on three different samples, under various conditions of normal load, was 10° which compares with a value of 13.5° obtained for the angle of internal friction from laboratory compression tests.

The stable bank referred to previously is a cut about 200 ft deep in the Cucaracha formation on the west bank of the canal, just south of Zion Hill, which has an average slope of 1 vertical on 2.7 horizontal. This bank has never been disturbed by slides but is composed of material similar to the Cucaracha material that would be encountered in the excavation of a sea-level canal. Computations were made to establish the cohesion needed for stability of the bank along the most critical sliding arc, assuming a factor of safety of 1.0 and a friction angle of 10°, as found from the laboratory tests. In this way a value for the cohesion of the formation as a whole was obtained which provides a reasonably sound basis for analysis of other cuts in similar material, since the effects of weak and strong bedded units, slickensides, joints, fractures, and gouge areas are automatically accounted for.

In the analyses, the ground-water table was considered to have been at the level of a bed of material known to provide good drainage, and therefore the effect of sudden drawdown, applied as a simple approximation of the forces of steady seepage, was limited to the mass below this bed. The analyses for a factor of safety of unity resulted in a value of 16 lb per sq in. (2,300 lb per sq ft) for the cohesion of the Cucaracha formation. This value was checked by an analysis of the first known shear slide that involved the Cucaracha formation— that of October, 1907—when Gaillard Cut was about 100 ft deep and presumably was being excavated to a slope of 3 vertical on 2 horizontal. The value has also been checked by analytical studies of the East Culebra and West Culebra slides.

The adopted strength values, cohesion equal to 16 lb per sq in., friction angle equal to 10°, and a factor of safety of 1.3, were used in developing the slope curve shown in Fig. 64. Sudden drawdown over the entire height of cut was assumed for the Cucaracha slopes. This assumption simplified the analyses and yet was somewhat conservative with regard to steady seepage out of the bank, a condition which might prevail after excavation. The part of this curve for cuts deeper than 200 ft was developed from data published by D. W. Taylor.[45] The part of the curve for cuts less than 200 ft deep is empirical and is based on recommendations made by Mr. MacDonald on the basis of his broad experience with the Cucaracha formation. Slopes for cuts less than 200 ft deep can justifiably be flatter than theoretically necessary, because the weakening effect of from 20 ft to 40 ft of weathered rock and residual clay overburden may be of some importance in shallower cuts.

[45] "Stability of Earth Slopes," by D. W. Taylor, *Journal*, Boston Soc. of Civ. Engrs., July, 1937, p. 197.

Atlantic Muck.—The extreme instability of the Atlantic muck was first observed in the early 1900's during the excavation for the north approach walls of the existing Gatun Locks. The muck was excavated to El. −55 on slopes of 1 vertical on 5 horizontal; but, before the dredging was completed, the material had slid in some places making the slopes as flat as 1 vertical on 13 horizontal. After the excavation was dewatered, when the pile foundation for the approach wall was about half completed, the east bank gave way and covered the greater part of the foundation with mud to a depth of from 6 ft to 18 ft. The final slope was 1 vertical on 20 horizontal.

In the design of the Third Locks, a similar slope problem was encountered in the excavation of an extensive muck area for the construction of the approach walls of the new Gatun Locks. Unconfined and triaxial compression tests, supplemented by direct-shear tests, were made on representative undisturbed samples of the material. The first tests on the muck were run quickly; the entire time of the test, after consolidation of the sample to overburden pressure had been completed, was only from 10 min to 20 min. It was later found that lower results were obtained if the tests were run at a slower loading rate. A critical loading rate of 1.6 lb per sq in. per 15 min was found to give the lowest strengths and was adopted for all subsequent tests. The duration of tests for that loading rate varied from 60 min to 300 min. The results of these tests showed conclusively that, independently of the time of loading, the strength of the muck depends on the degree of consolidation of the material at the time of shearing. Thus, within any muck deposit, the strength of the muck is greater at increasing depths, since the consolidating pressures increase with depth. Fig. 66 shows data from the final series of tests, made at the critical
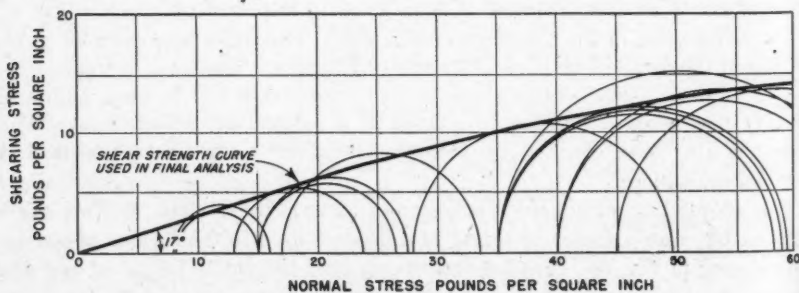

FIG. 66.—SHEAR STRENGTH CURVE FOR ATLANTIC MUCK

loading rate, which indicate the strength characteristics of the muck. (Each circle in Fig. 66 represents one triaxial compression test on a sample consolidated under pressures equal to or greater than the overburden loads and then loaded at the rate of 1.6 lb per sq in. per 15 min to failure.) In the Third Locks design studies for slopes in Atlantic muck to be cut by dredging and left permanently submerged, it was found that stability was influenced more by the height of the bank above permanent water level than by the depth of cut below water level—because the saturated weight of the muck above water level is about 90 lb per cu ft, whereas the weight of the submerged material is

only about 28 lb per cu ft. The proper slope for a permanently submerged cut in muck, therefore, is dictated largely by the height of the top of the cut above water. The slope adopted for use in the Gatun Locks area, where the top of muck was less than El. 10 and the total depth of cut was greater than 30 ft, was 1 vertical on 2.5 horizontal. Such slopes were actually cut during the period from 1941 to 1945 and have remained stable since that time.

A large part of the sea-level canal between the Atlantic entrance and Barro Colorado Island would require cuts in Atlantic muck which would be made by dredges operating on the present level of Gatun Lake. After lowering Gatun Lake to sea level, the top of the muck banks would be exposed above water level, in some areas as high as El. 35. Undisturbed samples of the muck underlying Gatun Lake were tested and found to have strength characteristics approximating those shown in Fig. 66.

One cross section for sea-level canal excavation through muck in the Gatun Lake area, Fig. 67, was designed to meet all conditions. This slope was ana-

FIG. 67.—EXCAVATION SLOPES IN MUCK

lyzed by the Swedish circular arc method, using the shear strength curve of Fig. 66, and the minimum factor of safety obtained was 1.3. The slope of 1 vertical on 2.5 horizontal below water level results from the desire to maintain as uniform a channel cross section as possible, in keeping with the cross section through the rock-cut areas, and from the successful experience with this particular slope in the Gatun north approach channel. The uniform slope of 1 vertical on 10 horizontal above water level would simplify the dredging and is amply safe for top-of-slope elevations as high as El. 35.

Other Rocks.—Slope-design standards for materials other than the Cucaracha and Atlantic muck formations were developed empirically. These can be classified into three groups: (1) Soft rocks—the Culebra, La Boca, and Las Cascadas formations; (2) medium rocks—the Gatun, Bohio, and Caimito formations; and (3) hard rocks—basalt, the Pedro Miguel conglomerate, and Bas Obispo formations. Slides in these materials, where encountered in the existing canal, were uncommon and usually resulted from structural causes.
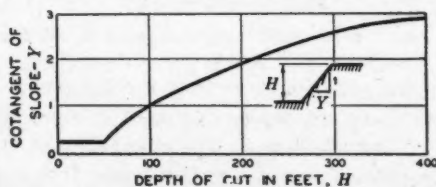

FIG. 68.—EXCAVATION SLOPE CURVE, CULEBRA FORMATION

The slope curve for the first and weakest of the three groups is shown in Fig. 68. This curve is based on recommendations made by Mr. MacDonald and on experience with previous excavation in the Culebra formation. A typical slope based on the curve in Fig. 68 is shown in Fig. 69.

Fig. 70 shows the standards for excavation slopes in the other two groups of materials, medium and hard rocks. The two standards differ only in the vertical interval between berms. The channel slope is 3 vertical on 2 horizontal up to El. 15. Above a 45-ft berm at this elevation the slopes are 12 vertical on
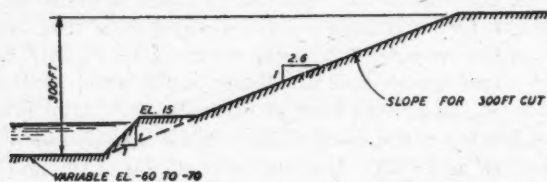

FIG. 69.—TYPICAL SLOPE BASED ON SLOPE CURVE IN FIG. 68

1 horizontal with 25-ft berms every 50 ft in elevation for the medium rocks, and every 100 ft for the hard rocks.

Considering stability only, it is possible that, in any of the medium or hard rocks, vertical cuts as high as 500 ft would stand without danger of major failures. However, rockfalls are always a possibility and would be particularly dangerous during construction. A 45-ft berm at El. 15 is provided to prevent

(*a*) BASALT, AGGLOMERATE, AND BAS   (*b*) GATUN, BOHIO, AND CAIMITO
OBISPO (HARD ROCKS)   (MEDIUM ROCKS)
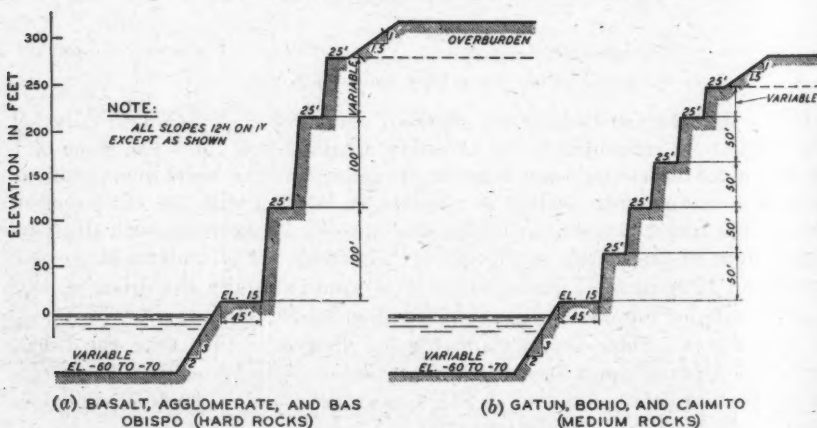
FIG. 70.—EXCAVATION SLOPES FOR MEDIUM AND HARD ROCKS

falling rock from reaching the channel. The 25-ft berms above El. 15 would provide a reasonable degree of safety during construction.

Channel Slopes.—The channel cross section should be as nearly uniform as possible to provide the best navigation conditions. Rock slopes for a sea-level canal designed only for stability would vary from 12 vertical on 1 horizontal for hard rocks to as flat as 1 vertical on 9.4 horizontal for the Cucaracha formation. To reduce this extreme variation for the benefit of navigation, the slopes below El. 15 were modified as follows: In hard rocks, the channel slopes would be 3 vertical on 2 horizontal; in Cucaracha and other soft rocks, a channel slope of 1 vertical on 1 horizontal would be used as indicated in Fig. 65 and 69.

Except for this modification, the over-all slope in these soft materials would be designed by the slope curves.

DYNAMIC LOADS

Large Explosions.—The effects of large bomb explosions in the vicinity of excavations were not taken into account in the development of the slope-design standards described previously. Investigations were made, however, of the effect that large explosions in air and on or under the ground might have upon the stability of slopes. Research was conducted by Harvard University to investigate the effects of explosive forces on the strengths of soils and rocks. This research is reported in the eighth Symposuim paper.

The studies indicate that sliding failures caused by dynamic loads on a statically safe slope would probably not result in closure of the canal. From present evidence, flattening the slopes beyond the requirements for static stability, therefore, is not believed to be necessary.

Earthquake Hazards.—According to information furnished by L. Don Leet, Professor of Geology at Harvard University, landslides do not occur at distances greater than 50 miles from the epicenter of the largest earthquakes known to have occurred, or at more than 20 miles from one as large as that in Japan in 1923. The computed accelerations which produced slides range from around 0.5 g to 0.7 g. The earthquake history of the Isthmian region around the Canal Zone indicates that no serious danger exists of an earthquake large enough and near enough to place the Canal Zone within such a landslide radius. The closest major active zone on record has been off the Los Santos peninsula, more than 100 miles from Balboa. Absolute prediction, either positive or negative, of the time and place of occurrence of earthquakes cannot be made. It is concluded from the study, however, that earthquake-induced landslides in cuts for the Panama Canal would be improbable; therefore, no allowance for earthquake forces has been made in the design of canal slopes.

APPLICATION OF SLOPE-DESIGN STANDARDS

The design standards described previously for excavation slopes under conditions of static loading are sufficiently detailed for quantity estimating and construction planning purposes. The application of these standards to the 4-mile section of a sea-level canal through the Continental Divide north of Pedro Miguel is illustrated in Fig. 71. Approximately 30% of the total required excavation is concentrated in this area. This section of the canal would also include the deepest cuts.

In the development of the standards illustrated by the slopes shown in Fig. 71, minimum strength values have been used for each of the geologic formations encountered. Before actual construction of a sea-level canal, more detailed geologic data would be available, and the slopes would be modified to take advantage of stronger phases of some formations. It is believed that any modification would result in a reduction of excavation quantities.
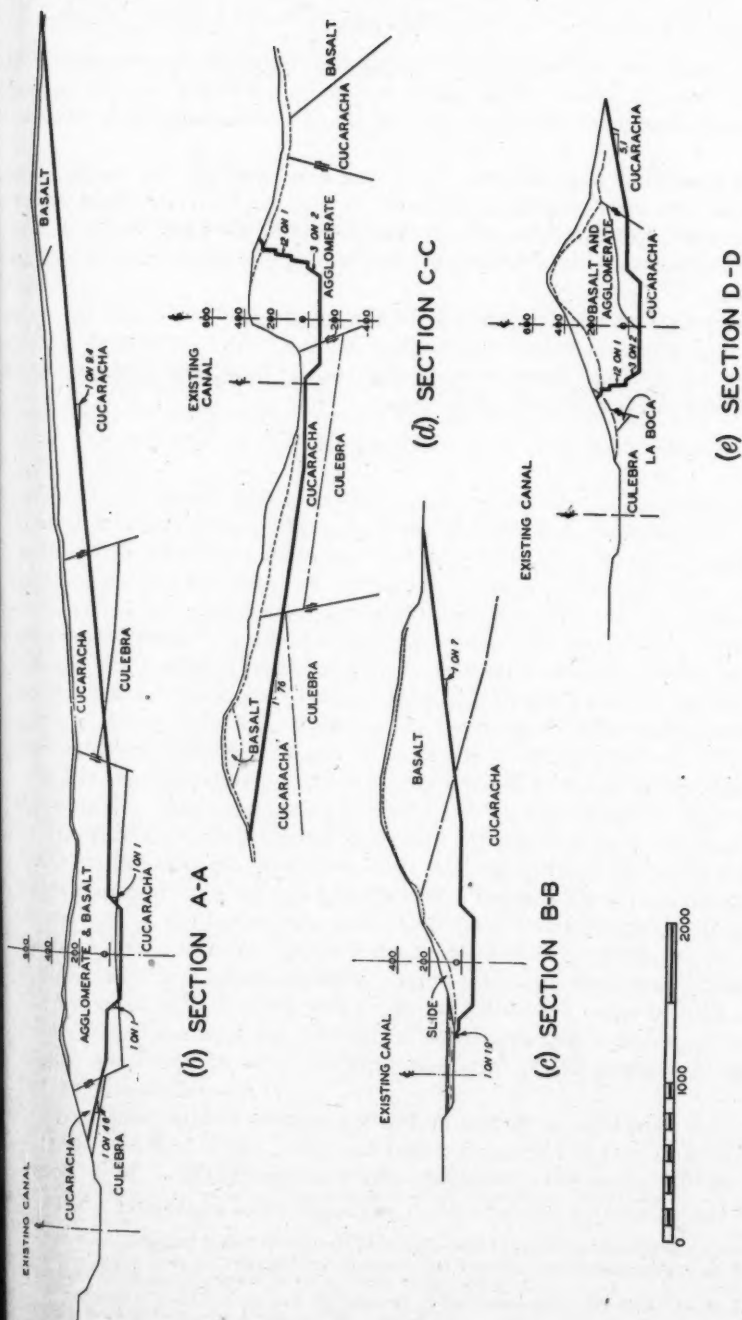
(a) PLAN
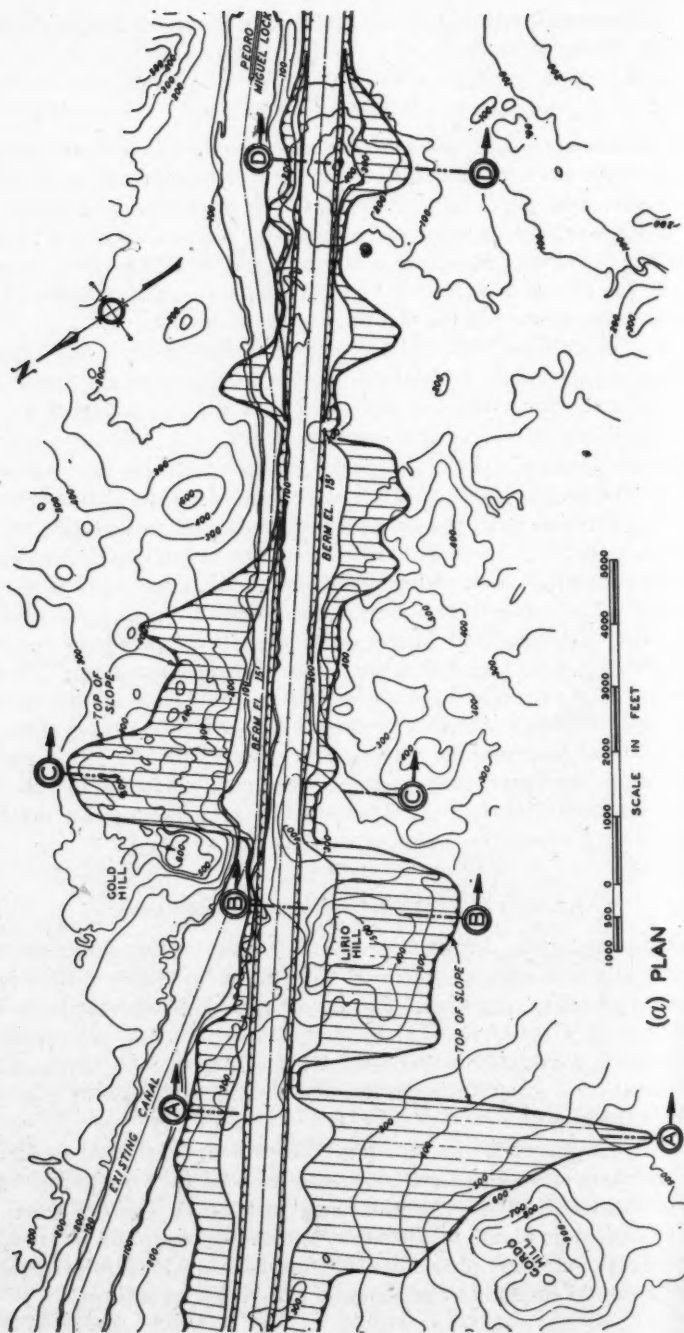
FIG. 71.—EXCAVATION LIMITS IN DEEP CUT SECTION

Conclusions

Although large slides occurred during and following construction of the present Panama Canal, major slides would be prevented in the proposed Panama sea-level canal by excavating the channel banks initially to the proper slopes.

The analytical methods and the testing employed for the design of safe slopes conform with established methods that had not been developed when the present canal was constructed. Studies of the slides that occurred during canal construction were of value in the design of stable slopes for the Cucaracha formation.

The design standards presented in this paper are considered to produce slopes that would be stable under static loading. Any slides that might be initiated by dynamic forces from earthquakes or large explosions would not be expected to cause closure of the canal.

STRENGTH OF SOILS UNDER DYNAMIC LOADS

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Assoc. M. ASCE

Synopsis

This paper describes apparatus developed and results of tests performed to investigate the strength characteristics of soils and soft rocks under dynamic loads.

Introduction

In connection with studies of the stability of slopes under the effects of bombing, laboratory investigations on the strength of soils and soft rocks under dynamic loading are being conducted at Harvard University for The Panama Canal.

This investigation is expected also to benefit other engineering problems in which soil is subjected to dynamic loading, such as the effects of earthquakes on dams and their foundations, or the effects of transient loading by fast moving traffic on airfield and highway pavements and the underlying materials.

Conventional strength tests on soils are either unconfined compression, or triaxial compression, or direct-shear tests. Any of these tests can be performed with either a controlled stress or a controlled strain loading apparatus. Loading of the specimen in such tests is performed over a period of at least several minutes. Such tests will be referred to herein as static strength tests, to distinguish them from the dynamic tests described in this paper.

It has been recognized that the strength of soil increases as the rate of loading increases. For example, in connection with the design of the third locks for The Panama Canal, a series of triaxial compression tests was conducted to determine the strength of undisturbed, soft organic clay by producing failure within a range of from 1.7 min to more than 7 hours. These tests indicated that the strength at the fastest rate of loading was about 40% greater than that at the slowest rate. D. W. Taylor,[48] Assoc. M. ASCE, investigated the strength of a clay that was remolded at the liquid limit and then consolidated under 4.22 kg per sq cm. Failure was produced within the range of from 4 min to 8 days. In these tests the strength of specimens that were loaded to failure quickly was found to be about 25% greater than the strength of specimens that were loaded slowly.

Investigations have been performed on metals to determine their strength at various rates of strain. One comprehensive series of tension tests was performed by M. J. Manjoine[49] on a mild steel within the range of from 1×10^{-6}

strain per sec to $1 \times 10^{+3}$ strain per sec, which corresponds to a range of time necessary to reach the ultimate strength of approximately 2.3 days to 0.0002 sec. The ultimate strength of specimens tested in the shortest time was found to be about 50% greater than that of specimens tested in the slowest time.

For fast transient tests on soils and soft rocks, it was necessary to develop apparatus for applying dynamic loads and for measuring and recording the loads applied and the resulting deformations of test specimens.

Apparatus for Applying Transient Loads

After a comprehensive review of dynamic testing apparatus developed for various purposes, the writers realized that none of these apparatus would be suited for this investigation, and they were obliged to develop new apparatus
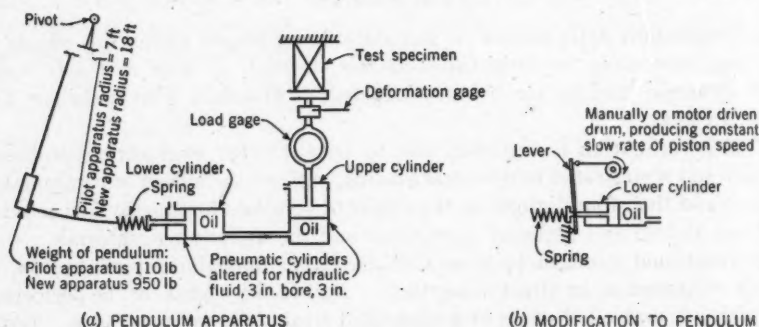

(a) PENDULUM APPARATUS   (b) MODIFICATION TO PENDULUM
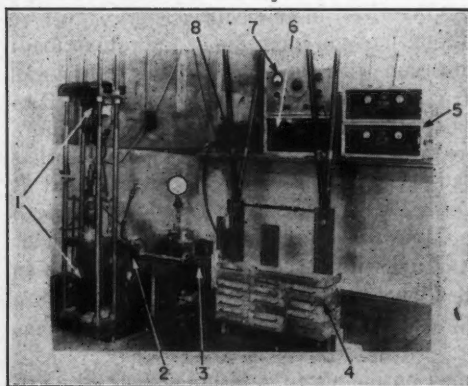
Fig. 72.—Pendulum Loading Apparatus

Fig. 73.—General View of Pendulum Loading Apparatus, with Recording Instruments in the Background

LEGEND

1. Loading frame (see Fig. 74)
2. Lower cylinder
3. Spring
4. Pendulum, weighted to 700 lb
5. Strain indicators
6. Power supply
7. Oscillator
8. Oscillograph

for applying transient loads. The type of loading desired was a transient load in which the test specimen is subjected to a rapid loading and unloading, simulating the effect of the first stress wave created by an explosion. As a criterion for the speed of load application, it was found convenient to define

the time of loading as the difference in time between the beginning of test and the time at which the maximum compressive stress is reached. The value for the fastest time of loading for use in this investigation was determined in consultation with H. M. Westergaard, M. ASCE, and L. Don Leet, special consultants to The Panama Canal. The value thus decided on was 1/100 sec. The time for the slowest loading was determined by the desire to overlap with the fastest loading time used in static strength tests.

Three different types of apparatus for applying transient loads in triaxial compression and unconfined compression tests were developed simultaneously,

LEGEND

1. Upper hydraulic cylinder and piston

2. Shielded cable to deformation gage

3. Deformation gage

4. Tilting cap

5. Reaction for deformation gage

6. Unconfined compression specimen

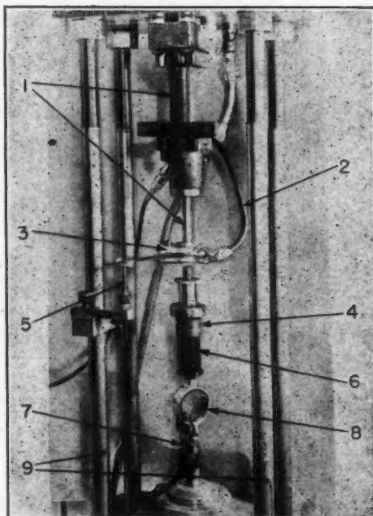7. Shielded cable to load gage

8. Load gage

9. Tie rods

FIG. 74.—CLOSE-UP OF LOADING FRAME OF THE PENDULUM APPARATUS, SHOWING THE ARRANGEMENT FOR THE UNCONFINED TRANSIENT COMPRESSION TEST

since it was not certain what method of load application would be best suited to this investigation. It finally was found that all three types were needed because they supplemented one another in the range of time of loading for which each type was best suited.

Pendulum Loading Apparatus.—Figs. 72 and 73 show a diagram and a photograph of the pendulum loading apparatus which utilizes the energy of a pendulum that is released from a selected height and strikes a spring connected to the piston rod of a 3-in. bore cylinder. This lower cylinder, in turn, is connected hydraulically to an upper cylinder of the same bore, which is mounted within a loading frame. Fig. 74 is a detailed view of the loading frame of the pendulum apparatus.

The time of loading for which this apparatus was found best suited ranges between 0.01 sec and 0.05 sec.

The Falling Beam Loading Apparatus.—The loading apparatus in Fig. 75 utilizes the unconfined compression test apparatus of the "universal soil loading machine"[50] for the application of a transient load.   The apparatus consists essentially of a beam with a weight and rider, a dashpot to control the velocity of the fall of the beam, and a yoke for transmitting the load from the beam to the specimen.   A small beam mounted above the yoke counterbalances the weight of the beam. This apparatus was found to be suited for a relatively long time of loading, ranging from 0.5 sec to about 300 sec.
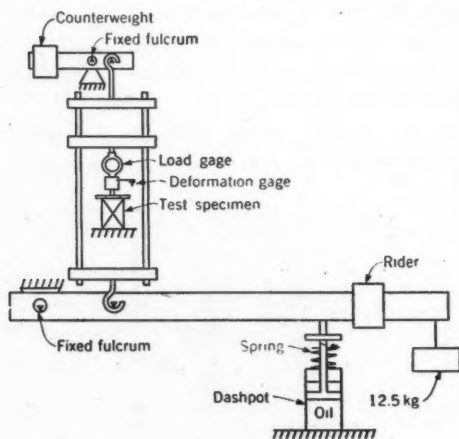

FIG. 75.—FALLING BEAM LOADING APPARATUS

The Hydraulic Loading Apparatus.—The loading apparatus in Figs. 76 and 77 consists of a constant volume vane-type hydraulic pump connected to a hydraulic cylinder through valves by which either the pressure in the cylinder or the volume of liquid delivered to the cylinder can be controlled.   The peak load that can be produced with this
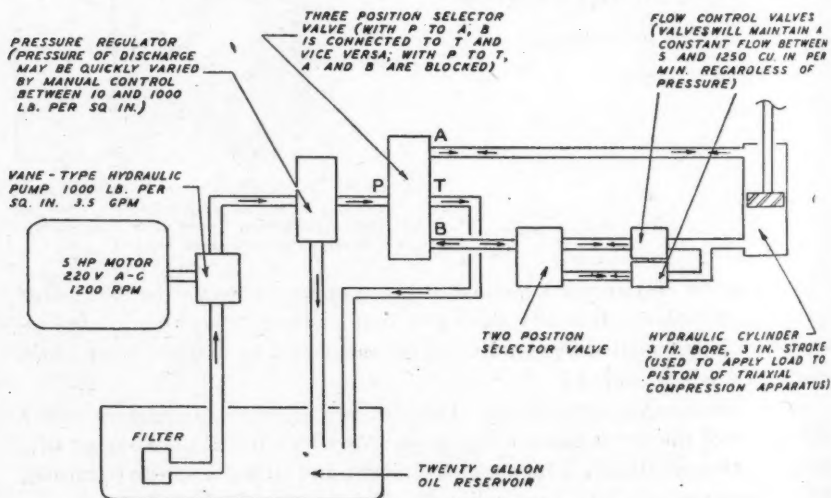

FIG. 76.—DIAGRAM OF HYDRAULIC LOADING APPARATUS

apparatus is much greater than can be obtained by either the falling beam or the pendulum types of loading apparatus.

[50] "Recent Developments in Soil Testing Apparatus," by P. C. Rutledge, in "Contributions to Soil Mechanics 1925-1940," Boston Soc. of Civ. Engrs., Boston, Mass., 1940, pp. 243-250.

This apparatus is used for testing soft rocks from the Canal Zone with a time of loading between 0.05 sec and any desired slow loading.
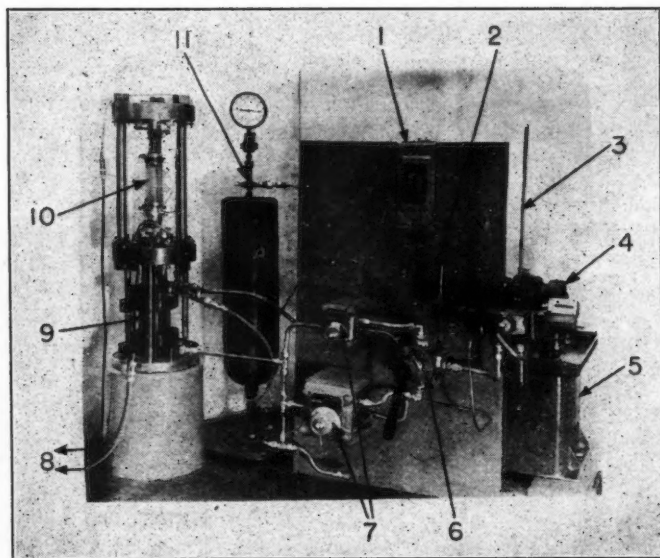
LEGEND

Point	Description	Point	Description
1	Starter switch for 5-hp electric motor which is behind panel	7	Flow control valves
2	Three-position selector valve	8	Cables from load and deformation gages (inside pressure chamber) leading to recording instruments
3	Handle of pressure regulator		
4	Vane-type hydraulic pump operated by electric motor	9	Hydraulic cylinder
5	Oil reservoir	10	Test specimen inside transparent pressure chamber
6	Two-position selector valve	11	Air pressure reservoir

FIG. 77.—PHOTOGRAPH OF HYDRAULIC LOADING APPARATUS

APPARATUS FOR APPLYING STATIC LOADS

Two types of loading apparatus were used for determining the static compressive strength.

The Fairbanks Scale Loading Apparatus.—This type consists of a conventional 500-lb Fairbanks platform scale equipped with a loading yoke and a mechanical jack.[51] It is used extensively in performing soil tests with stress control.

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[51] "The Soil Mechanics Laboratory at Harvard University," by P. C. Rutledge *Proceedings*, International Conference on Soil Mechanics and Foundation Eng., June, 1936, Vol. II, pp. 85–97, particularly Figs. 14 and 15.

The Hydrostatic Loading Apparatus.—This type consists essentially of the two hydraulic cylinders of the pendulum apparatus and a motor-driven drum and lever with which the travel of both hydraulic pistons can be controlled at a constant, slow rate.

TRIAXIAL AND UNCONFINED COMPRESSION APPARATUS

All triaxial compression apparatus used in this investigation for transient load tests have been adapted from triaxial apparatus previously constructed at Harvard University for static testing.[52] One of these is a vacuum type and three are compression types. The vacuum type of apparatus is limited to

Loading yoke of falling beam apparatus

Fitting to replenish castor oil in piston packing

Deformation gage

Piston clamp

Leads with cable connectors

Piston packing

Lucite cylinder

Spiral of shielded lead wires from load gage

Loading piston

Air

Load gage

Castor oil

Cap

Spiral of thin Saran tubing

Test specimen

Compression chamber pressure inlet
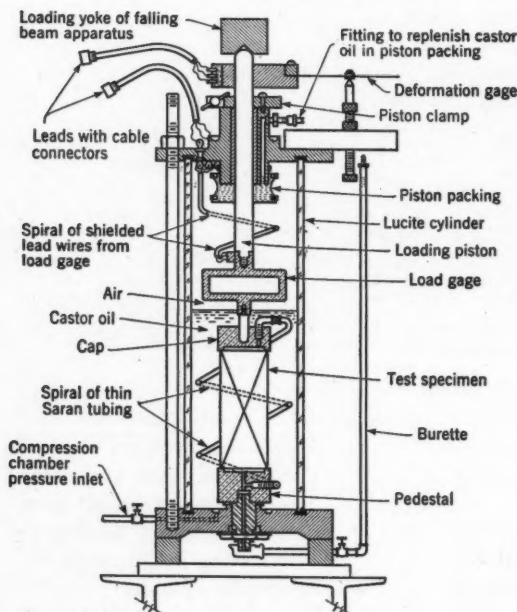
Burette

Pedestal

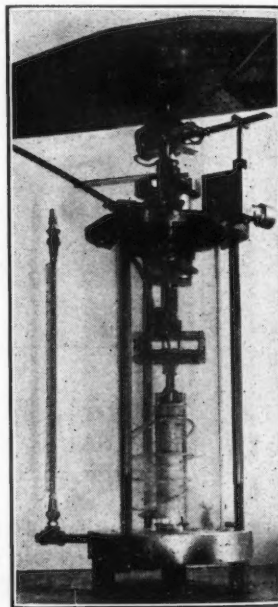FIG. 78.—DIAGRAM OF TRIAXIAL TRANSIENT COMPRESSION APPARATUS

FIG. 79.—PHOTOGRAPH OF TRIAXIAL COMPRESSION APPARATUS

tests on dry, noncohesive soils under a minor principal stress less than one atmosphere. The compression-type apparatus illustrated in Figs. 78 and 79 is suitable for tests on either cohesive or noncohesive soils under a minor principal stress up to about 6 kg per sq cm.

Figs. 72, 73, and 74 show the pendulum apparatus assembled for unconfined compression tests; and Fig. 75 shows the falling beam apparatus similarly assembled.

APPARATUS FOR MEASURING AND RECORDING TRANSIENT LOADS AND DEFORMATIONS

A comprehensive study was made of instruments available for measuring and recording transient compressive forces and deformations. In addition,

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[52] "Progress Report on Triaxial Shear Research and Pressure Distribution Studies on Soils," U. S. Waterways Experiment Station, Vicksburg, Miss., April, 1947, pp. 130–154.

several months of intensive work were required to develop suitable load and deformation gages.   To measure and record rapidly changing loads and deformations, metal-electric (SR-4) strain gages with companion strain indicators and oscillographs were found most suitable, in part because such equipment was readily available.   A brief description of this instrumentation follows:

Load Gage.—For measuring load, a load gage of rectangular or cylindrical shape is used, with four metal-electric (SR-4) strain gages mounted on the inside face.   (The irregular mass on the interior surfaces of the load gage in Fig. 80 is wax placed there for the protection of the SR-4 gages.)   Load gages are also shown in Figs. 72, 74, 75, 78, and 79.

Deformation Gage.—For measuring deformation, a thin flexible steel spring cantilever is used (Fig. 81) with metal-electric (SR-4) strain gages mounted on
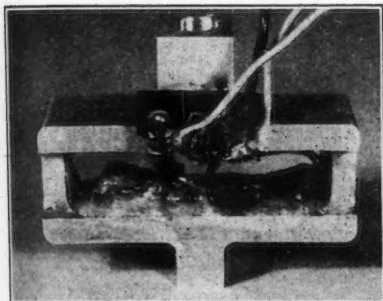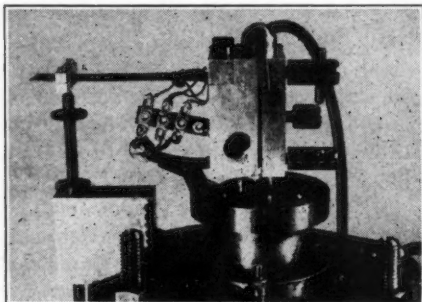

FIG. 80.—LOAD GAGE   FIG. 81.—DEFORMATION GAGE

the cantilever, the base of which is clamped to the loading piston.   The tip of the cantilever reacts against an adjustable screw which is mounted on the head plate of the triaxial compression apparatus.   Deformation gages are also shown in Figs. 72, 74, 75, 78, and 79.

Recording Apparatus.—Equipment for amplifying and recording the signal produced by the SR-4 strain gages consists of two strain indicators,[53] an oscillator, a power supply, and oscillographs.   The strain indicators contain two arms of a Wheatstone bridge and a carrier type amplifier.   The other two arms of the Wheatstone bridge are the SR-4 strain gages on the load or deformation gages.   This equipment can be seen in the background of Fig. 73.

MATERIALS TESTED AND TECHNIQUE OF TESTING

Static and transient compression tests have been performed to date on the following materials:

(1) *Manchester (N. H.) Sand.*—This is a clean, medium sand obtained by screening from a glacial-fluvial deposit and contains only the fraction between 0.42 mm and 0.21 mm.   It consists principally of subangular quartz grains, and has a void ratio in the densest state of about 0.61 and in the loosest state of about 0.88.

53 "A Carrier Type Strain Indicator," by George W. Cook, *Report No. 565*, David Taylor Model Basin, U. S. Navy, Washington, D. C., November, 1946.

(*2*) *Cambridge* (*Mass.*) *Clay.*—This is a medium soft, inorganic clay, with occasional thin silt partings, brittle in the undisturbed state and soft and sticky when remolded. Its natural water content ranges from 30% to 50%. For layers of this clay having a natural water content of from 40% to 50%, the liquid limit was found to range from 44 to 59 and the plastic limit from 21 to 27. For layers of this clay having a natural water content of from 30% to 40%, the liquid limit was found to range from 37 to 44 and the plastic limit from 20 to 23.

(*3*) *Boston* (*Mass.*) *Clay.*—This is similar, geologically and in its appearance, to the Cambridge clay. The samples tested had a natural water content between 32% and 36%, an average liquid limit of 42 and a plastic limit of 20.

(*4*) *Stockton* (*Calif.*) *Clay.*—This is a tough, brown clay, locally called adobe. The sample tested was obtained from the compacted fill on which the Stockton pavement traffic test[54] was conducted, and was about 90% saturated, with a natural water content of about 25%, a liquid limit of from 60 to 64, and a plastic limit of from 20 to 23.

(*5*) *Atlantic Muck* (*Canal Zone*).—This is an organic clay, having a natural water content ranging between 50% and 135%, a liquid limit of from 55 to 95, and a plastic limit of from 30 to 55. The samples tested were obtained in 3-in. diameter, thin-walled tubes from a boring into a natural deposit located adjacent to the Panama Canal.

(*6*) *Cucaracha Shale* (*Canal Zone*).—This is a slickensided clay-shale. The samples tested were obtained from core borings in the Cucaracha formation located in the vicinity of the Gaillard Cut, Panama Canal.

Details of Testing.—The Cambridge and Boston clays were tested both in unconfined compression and in triaxial compression, whereas the Stockton clay was tested in triaxial compression only. Cucaracha specimens were tested in triaxial compression and Atlantic muck in unconfined compression. Manchester sand was tested in the dense state, dry, in a vacuum-type triaxial compression apparatus.

Most unconfined compression test specimens of Cambridge clay were 6.3 cm square and about 16 cm high; all triaxial compression test specimens of clay were 3.56 cm in diameter and about 9 cm high. Unconfined compression test specimens of Atlantic muck were 2.5 cm square and about 6 cm high. Triaxial compression test specimens of Cucaracha shale were 5 cm in diameter and 12 cm high, and specimens of Manchester sand were 7.1 cm in diameter and about 18 cm high.

Two types of triaxial compression tests were performed, designated "quick" and "consolidated-quick." A quick triaxial compression test is one in which there is no preliminary consolidation and no drainage of pore water during the test. A consolidated-quick triaxial compression test is one in which the specimen is allowed to consolidate under a hydrostatic pressure, but, during the subsequent quick axial loading, there is practically no drainage of pore water. A detailed discussion of these types of tests appears elsewhere.[55]

[54] "Flexible Pavement Test Section for 300,000-Lb. Airplanes, Stockton, California," by Ralph A. Freeman and O. J. Porter, *Proceedings*, Highway Research Board, National Research Council, Vol. 25, 1945, pp. 23–44.

[55] "Progress Report on Triaxial Shear Research and Pressure Distribution Studies on Soils," U. S. Waterways Experiment Station, Vicksburg, Miss., April, 1947.

The triaxial compression test specimens were surrounded by two rubber membranes, each averaging approximately 0.05 mm thick. Specimens of clay and Cucaracha shale for consolidated-quick triaxial compression tests were fully consolidated under a lateral pressure of either 3 kg per cm² or 6 kg per cm² before axial compression was started. Manchester sand specimens were tested under a lateral pressure of 0.3 kg per cm² and 0.9 kg per cm².

Two procedures for performing transient compression tests were used, which are described below:

Controlled Impulse Method.—The specimen is subjected to a controlled impulse. The peak load exerted on the test specimen and its deformation depend on the stress-deformation and strength characteristics of the specimen. Typical stress-time and strain-time diagrams for an unconfined transient compression test on clay, using the controlled impulse method, are shown in Fig. 82.
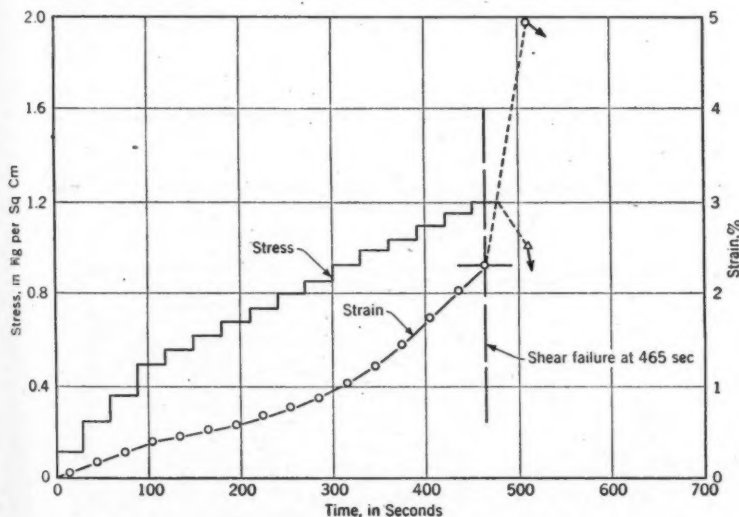

FIG. 82.—TIME CURVES FOR STRESS AND STRAIN; UNCONFINED TRANSIENT COMPRESSION TEST ON CAMBRIDGE CLAY

Controlled Strain Method.—The specimen is subjected to a rate of strain which is maintained approximately constant from the beginning of the test to the desired maximum strain. The peak load exerted on the specimen and the time of loading depend on the stress-deformation and strength characteristics of the specimen.

RESULTS OF TESTS

Representative results of a transient and a static unconfined compression test on Cambridge clay are shown in Figs. 82, 83, and 84. Figs. 82 and 83 are time curves for stress and strain and Fig. 84 shows the stress-strain curves. (The plotted points shown in Fig. 83 were taken from a continuous oscillograph record.)

All unconfined compression specimens of Cambridge clay failed with one or more clearly visible shear planes. To measure the slope of the shear planes, seven transient tests, with a time of loading of about 0.02 sec, and four static tests were so conducted that loading was stopped as soon after the shear planes
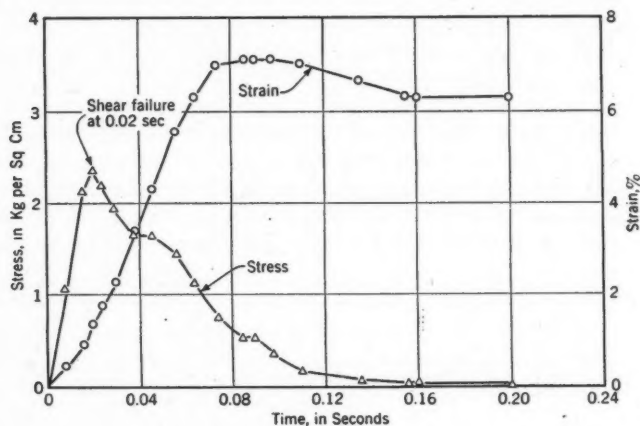

FIG. 83.—TIME CURVES FOR STRESS AND STRAIN; UNCONFINED STATIC COMPRESSION TEST ON CAMBRIDGE CLAY
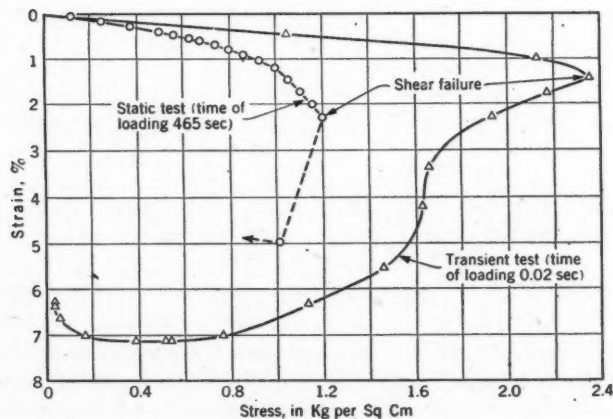

FIG. 84.—STRESS-STRAIN CURVES; UNCONFINED COMPRESSION TESTS ON CAMBRIDGE CLAY

appeared as was technically possible. The strain at the end of these tests ranged between 2% and 3% and it is believed that the distortion of the shear planes in these tests was negligible. The angles between the shear planes and the horizontal (plane of major principal stress) were carefully measured to determine whether there was a significant difference in these angles between static and fast transient loading. For the static tests, the angles ranged between 53°
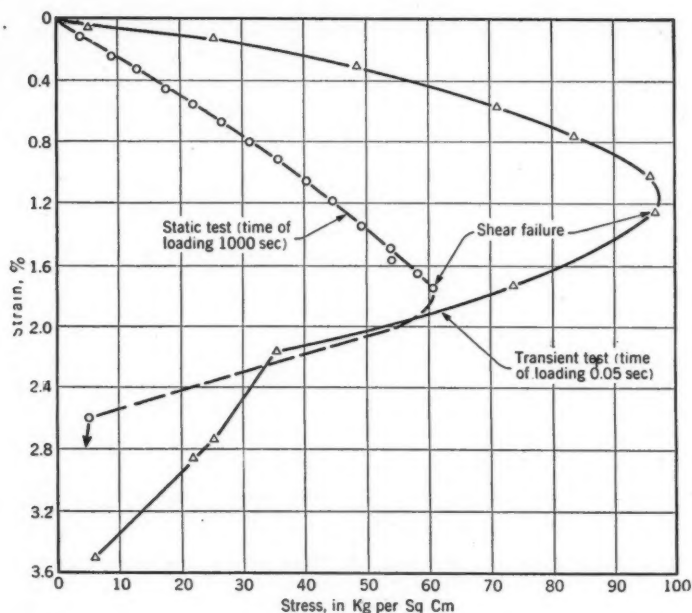
FIG. 85.—STRESS-STRAIN CURVES FOR TRANSIENT AND STATIC TRIAXIAL COMPRESSION
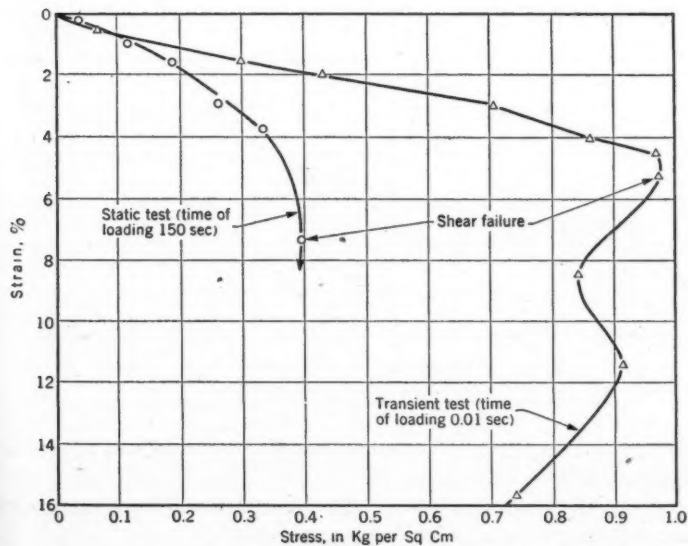TESTS (CONSOLIDATED-QUICK) ON CUCARACHA SHALE
($\sigma_3 = 6$ KG PER SQ CM)

FIG. 86.—STRESS-STRAIN CURVES FOR TRANSIENT AND STATIC UNCONFINED
COMPRESSION TESTS ON ATLANTIC MUCK

and 68°, with an average of 61°; and for the transient tests, between 47° and 65°, with an average of 58°.

Stress-strain curves for a transient and a static triaxial compression test on Cucaracha shale are shown in Fig. 85. This material failed along one or more clearly defined shear planes sloping from 35° to 65°. Usually the position
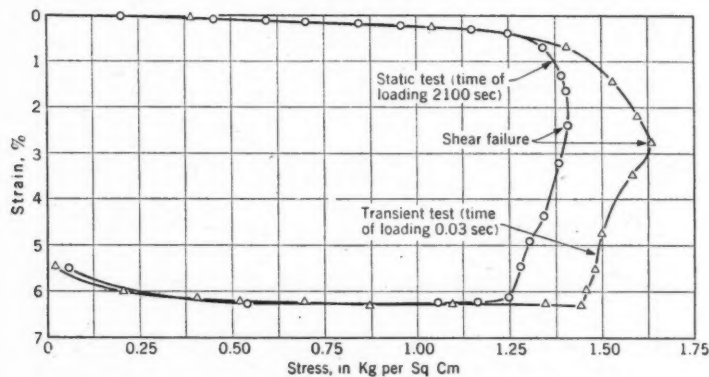
FIG. 87.—STRESS-STRAIN CURVES FOR TRANSIENT AND STATIC TRIAXIAL COMPRESSION TESTS ON MANCHESTER SAND
($\sigma_3 = 0.3$ KG PER SQ CM)
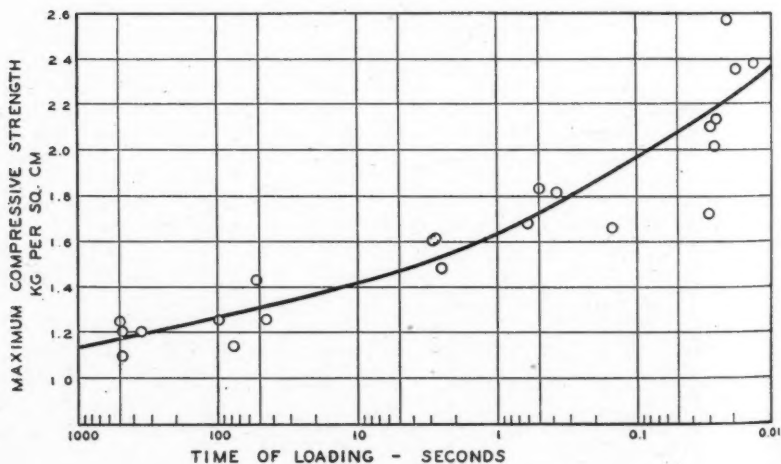
FIG. 88.—UNCONFINED COMPRESSION TESTS ON CAMBRIDGE CLAY

and slope of the shear planes were influenced by jointing and the presence of weaker phases of the Cucaracha shale. Both specimens tested represent a strong phase of Cucaracha shale. The points shown for the transient test were taken from a continuous oscillograph record.

Stress-strain curves for a transient and a static unconfined compression test on Atlantic muck are shown in Fig. 86. This material usually failed along one

or more clearly defined shear planes sloping about 50°. Some specimens failed by bulging with vertical tension cracks.

Stress-strain curves for a transient and a static vacuum-type triaxial compression test on Manchester sand are shown in Fig. 87. Both of these tests
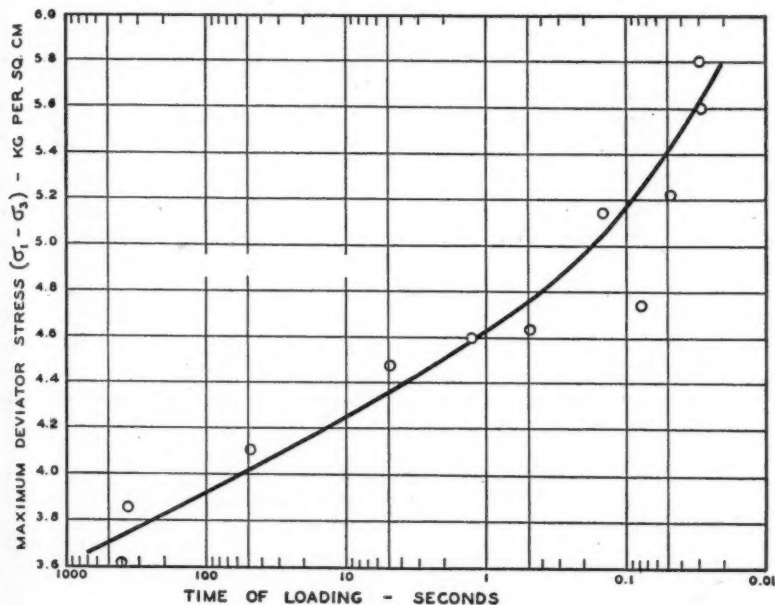
FIG. 89.—TRIAXIAL COMPRESSION TESTS ON CAMBRIDGE CLAY
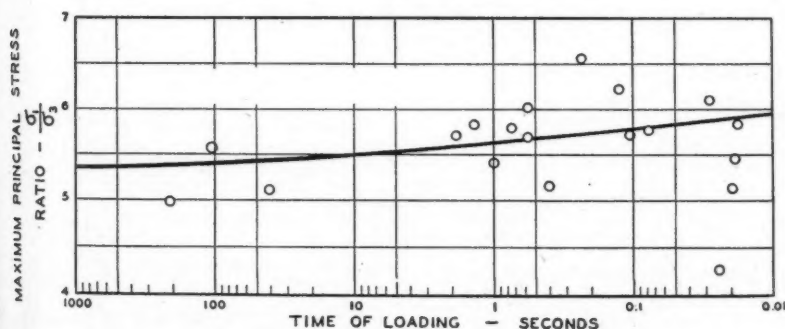($\sigma_3 = 6$ KG PER SQ CM)

FIG. 90.—TRIAXIAL COMPRESSION TESTS ON SAND
(VOID RATIO $= e_p = 0.620$; VACUUM PRESSURE $-P_V = 0.30$ KG PER SQ CM)

were performed in a vacuum-type triaxial compression apparatus on dense sand ($e = 0.61$). The points shown for the transient test were taken from the continuous oscillograph record. Failure of all specimens was by bulging without noticeable shear planes.

The relation between compressive strength and time of loading for a given material is determined from a series of static and transient compression tests. Such relationships are shown by Figs. 88 and 89 for Cambridge clay, and by Fig. 90 for Manchester sand.
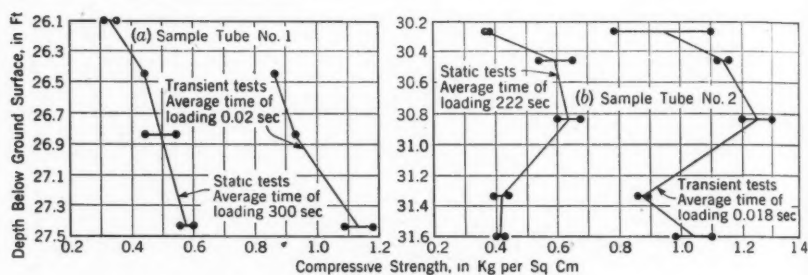

FIG. 91.—COMPRESSIVE STRENGTH VERSUS TIME OF LOADING FOR ALL TEST SERIES ON CLAY

Fig. 91 is a summary sheet showing the relationships between compressive strength and time of loading as obtained from all the tests on clays performed to the time this paper was prepared. An explanatory legend for Fig. 91 is given in Table 31.

TABLE 31.—LEGEND FOR FIG. 91

Curve No.	Sample No.	Source of clay[a]	Type of test	Water content at time of test[b] (%)
1	HP-6-C	Cambridge clay which was slowly dried to slightly higher than its shrinkage limit	Unconfined compression	27
2	HP-6-C	Cambridge clay which was slowly dried from its natural water content of about 47% to about 33%	Unconfined compression	33
3	HP-6-B	Cambridge clay	Consolidated-quick triaxial compression $\sigma_3 = 6$ kg per cm²	24 to 32
4	HP-6-A	Cambridge clay	Consolidated-quick triaxial compression $\sigma_3 = 6$ kg per cm²	32 to 35
5	HP-6-D	Cambridge clay	Consolidated-quick triaxial compression $\sigma_3 = 6$ kg per cm³	40 to 44
6	HP-6-C	Cambridge clay which was slowly dried from its natural water content of about 47% to about 37%	Unconfined compression	37
7	HP-7-A	Stockton clay	Quick triaxial compression $\sigma_3 = 3$ kg per cm²	24 to 27
8	HP-5-A	Boston clay	Quick triaxial compression $\sigma_3 = 6$ kg per cm²	32 to 35
9	HP-2	Cambridge clay	Unconfined compression	34 to 39
10	HP-6-B	Cambridge clay	Unconfined compression	35 to 40
11	HP-6-A	Cambridge clay	Unconfined compression	40 to 42

[a] All specimens were tested undisturbed. [b] For curves Nos. 1 to 6, the specimens were reduced in water content from their original by consolidation or drying. For curves Nos. 7 to 11, the specimens were tested at their original water content.

For the weakest clays the transient strength for the fastest time of loading was about twice the 10-min static strength. The ratio between these two values appears to decrease with increasing strength of the material. For the strongest clays included in Fig. 91, the fast transient tests showed a strength that was only about 50% greater than the 10-min static strength. Sufficient data are not available to plot the corresponding relationships for Cucaracha shale and Atlantic muck.

The stress-strain curves for Cucaracha shale in Fig. 85 indicate that the strength in a transient test, with a time of loading of 0.05 sec, is about 60% greater than that for a 10-min static test.
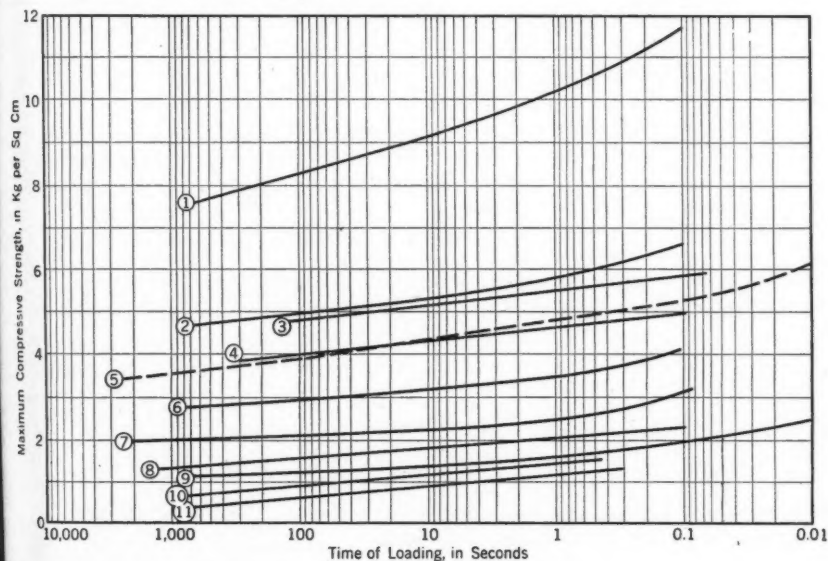
Sample tube No.	Average Time of Loading (Sec)		Average Modulus of Deformation (Kg per Sq Cm)	
	Static tests	Transient tests	Static tests	Transient tests
1	300	0.02	8.1	16.0
2	222	0.019	8.7	19.4

Fig. 92.—Unconfined Compression Tests on Atlantic Muck

Results of unconfined compression tests on Atlantic muck are summarized in Table 32. Fig. 92 shows profiles of results for two sample tubes. These tests indicate that the transient strength for a time of loading of about 0.02 sec is about twice the 10-min static strength.

Modulus of Deformation.—The original purpose of the investigation did not include the determination of the stress-deformation characteristics. Therefore, precise measurements of the deformation at small loads in transient tests have not yet been made. However, all tests have been analyzed as well as

possible for the purpose of arriving also at some information regarding the effect of time of loading on the initial slope of the stress-deformation curve as represented by the modulus of deformation. For the purpose of this investigation, the modulus of deformation is defined as the slope of a line drawn from the

TABLE 32.—RESULTS OF UNCONFINED COMPRESSION TESTS
ON ATLANTIC MUCK

Tube No.	Depth (ft) below ground surface	Water content[a] (%)	Static compressive strength[b] (kg per cm²)	Transient compressive strength[c] (kg per cm²)
1	24.1 to 28.1	109	0.31 / 0.35	....[d]
		78	0.44	0.87
		77	0.54 / 0.44	0.94
		85	0.55 / 0.60	1.18 / 1.09
2	28.1 to 32.1	67	0.38 / 0.37	0.69 / 1.00
		63	0.54 / 0.65	1.12 / 1.16
		59	0.60 / 0.67	1.30 / 1.20
		60	0.44 / 0.39	0.89 / 0.86
		52	0.40 / 0.43	0.98 / 1.10
4	35.4 to 39.2	120	0.33 / 0.33	....[d]
		118	0.26 / 0.20	....[d]
		121	0.37 / 0.31	0.64 / 0.91
		123	0.40 / 0.43	1.00 / 0.95
		122	0.47 / 0.53	....[d]
6	43.1 to 46.8	114	0.18 / 0.18	....[d]
		134	0.42 / 0.45	0.89 / 0.55
		135	0.43 / 0.34	0.68 / 0.88

[a] Water contents are average values for the static and transient tests for which the strength values are shown in the same horizontal column. [b] Time of loading ranging between 3 min and 9 min. [c] Time of loading approximately 0.02 sec. [d] Recording instruments failed to function satisfactorily.

origin through the point on the stress-deformation curve corresponding to a stress of one half of the strength.

The order of magnitude of the modulus of deformation, as obtained from the tests performed on clay and Cucaracha shale samples, is summarized in Table 33.

From Table 33, it is concluded tentatively that, for these cohesive materials, the modulus of defromation for fast transient loading is about twice the value for 10-min static loading.

TABLE 33.—MODULUS OF DEFORMATION OF CLAYS AND CUCARACHA SHALE

Material	Type of test	AVERAGE VALUES OF MODULUS OF DEFORMATION (KG PER CM²)	
		Static tests	Fast transient tests
Cambridge clay.....	Unconfined compression	200	400
	Consolidated-quick triaxial ($\sigma_3 = 6$ kg per cm²)	650	1,300
Boston clay.........	Quick triaxial	250	500
Stockton clay.......	Quick triaxial	250	500
Atlantic muck......	Unconfined compression	9	18
Cucaracha shale.....	Consolidated-quick triaxial ($\sigma_3' = 6$ kg per cm²)	5,000	12,000

As can be noted in Figs. 82 to 86, the slope of the stress-strain curves for transient tests appears to be almost constant to 50% of the failure load. Hence, the order of magnitude of the modulus of deformation for transient tests is independent of the stress, provided a reference stress of not more than one half of the compressive strength is selected. For this reason the ratio of the moduli for transient and static tests would remain about the same, even if the reference stress for a transient test is assumed equal to one half of the static strength.

The modulus of deformation of dry Manchester sand had the same order of magnitude in transient tests as in static tests, as shown by the nearly identical slopes of the stress-strain curves, Fig. 87.

CONCLUSIONS

The findings presented in this paper can be summarized as follows:

(1) The strength of the clays and the Cucaracha shale loaded to failure in about 0.02 sec was found to be between 1.5 and 2.0 times greater than their 10-min static strengths.

(2) The strength of sand increases only slightly with decreasing time of loading. The strength for the fastest time of loading (0.02 sec) was about 10% greater than that for 10-min static tests.

(3) The modulus of deformation of clay and Cucaracha shale for fast transient tests (time of loading about 0.02 sec) was found to be approximately twice that for 10-min static tests.

(4) The modulus of deformation of sand was found to be independent of the time of loading.

CONSTRUCTION PLANNING AND METHODS

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Synopsis

The principal construction features of the proposed Panama sea-level canal are described in this paper, including the various plans for the conversion of existing lock canal to sea level, and the construction methods on which the cost estimates are based.

Conversion of the existing canal to sea level in a single stage by deep dredging is preferable to conversion by stage dredging because it would be cheaper, would require a shorter construction period, and would interfere less with canal traffic.

For purposes of construction planning and estimates, the bulk of the dry excavation would be performed by large shovels or draglines of at least 25-cu-yd capacity, loading directly into dump scows which would haul the spoil to disposal areas in Gatun Lake. The wet excavation would be performed by conventional dredges supplemented by special hydraulic and bucket-ladder dredges capable of dredging to a depth of 145 ft below water surface. These methods of dry excavation and wet excavation are the most feasible and economical, and can be adapted most readily to the over-all project.

Introduction

Public Law No. 280 which authorized the Isthmian Canal Studies—1947 directed "a comprehensive review and study, with approximate estimates of costs, of means of increasing the capacity and security of the Panama Canal * * * [and] study [of] * * * a canal or canals at other locations * * *."

Compliance with Public Law No. 280 required initially the following studies:

1. Selection of general construction techniques and estimating methods to obtain approximate and comparable estimates of cost for the many canal routes to select the routes offering the best possibilities for development; and

2. Determination of the most practicable construction procedures and equipment, and preparation of detailed and dependable comparative estimates of costs of the best canal plans, to assist in the final selection of route and type of canal.

These procedures led to the selection of the Panama sea-level canal as the most economical means of increasing capacity and security to meet the future needs on interoceanic commerce and national defense. After the selection of

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the sea-level canal at Panama, more detailed construction studies and estimates were made.   The main features of these studies were:

(1) Investigation of new and special equipment for converting the Panama Canal to sea level; and

(2) Investigation of the major construction problems involved in executing the recommended plan of converting the present Panama Canal to sea level in a single stage by the "deep-dredging" method.

For the initial construction studies under Public Law No. 280, thirty possible routes or alinements (Fig. 6) were examined, of which eight were found on map inspection and, in some cases, on field reconnaissance, to offer the best possibilities for development.   Cost estimates for the more favorable of the sea-level and lock canals on eight routes were prepared by using excavation quantities obtained from center-line map profiles and, in each case, approximate classifications of materials.   Detailed investigations were made of the problems involved in the construction of both a lock canal and a sea-level canal in the Canal Zone and the immediate vicinity, and estimates of costs based on standard construction procedures were prepared for use in comparative analyses. Unit costs for excavation were those developed for the Panama route.   Costs of structures, such as locks, were developed for each route on the basis of height or lift, using the cost of locks at Panama as the estimating standard.   As shown by Tables 1 and 2, the Panama route is the most economical for both a sea-level canal and a lock canal.

After selection of the Panama sea-level canal, detailed studies and cost estimates were made of the most efficient and economic construction methods. The investigations and studies used as the basis for the estimates of cost for the construction of the Panama sea-level canal are described in this paper.

FEATURES OF THE MAIN CHANNEL

The Panama sea-level canal would have a channel 600 ft wide at a 40-ft depth, with a total depth below mean sea level varying uniformly from 60 ft at the Atlantic entrance to 70 ft at the Pacific end.   The principal features of the canal are shown in Fig. 7.   The total length of the canal from deep water in the Atlantic to deep water in the Pacific would be approximately 46 miles. The alinement would follow, generally, that of the present canal.   Sharp curves would be eliminated and tangents lengthened to meet navigation requirements. The sea-level canal alinement would be contiguous to the present canal in the section of deep cut through the Continental Divide and would permit the removal of a large part of the required rock excavation more economically by dry methods.   The alinement would utilize the excavation accomplished for the Third Locks structures and the approach channels at Miraflores and Gatun. The total required excavation is approximately 1,070,000,000 cu yd, of which 750,000,000 cu yd could be excavated in the dry.   Dredging would be required for the removal of the remaining 320,000,000 cu yd.   For comparison, the channel excavation by the United States to complete the existing canal totaled approximately 120,000,000 cu yd of wet and 125,000,000 cu yd of dry.   The

total usable channel excavation, including that done by the French, was approximately 275,000,000 cu yd.

The maximum depth of cut to be excavated would be 650 ft; however, the length of canal at this depth would be less than ½ mile.   Approximately 3½ miles of canal would be in cut deeper than 300 ft; 7 miles, in cut between 200 ft and 300 ft; 9 miles, in cut between 100 ft and 200 ft; and the remaining 26 miles, in cut less than 100 ft deep.   Of this 26 miles, 10 miles would be in a muck formation, a soft overburden material in which the maximum depth of cut would be less than 80 ft.

For convenience in estimating and for facilitating orderly completion of the excavation, the channel excavation has been divided into ten parts.   The material to be excavated has been classified into the following four types according to the relative difficulties of excavation: Common, soft rock, medium rock, and hard rock, as shown in Table 34.   The linear distribution of the material is shown in Fig. 93.
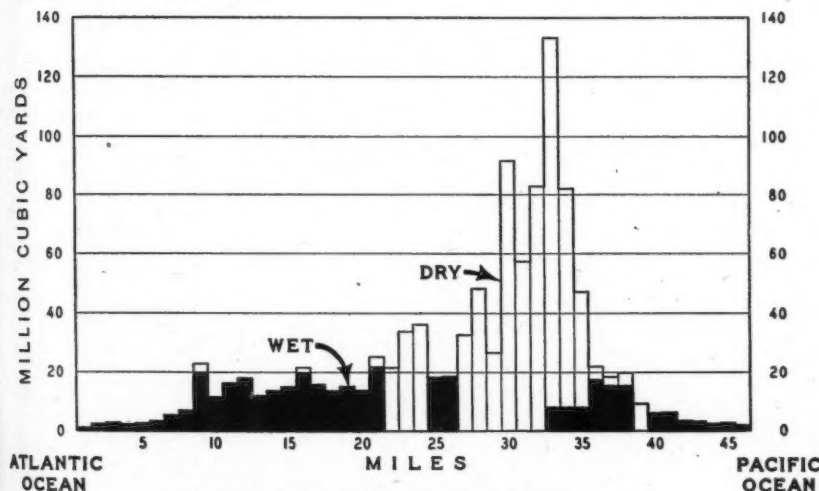
FIG. 93.—DISTRIBUTION OF EXCAVATION QUANTITIES

STRUCTURAL FEATURES

Tidal-Regulating Structures.—These structures consist of a tidal lock, a navigable pass, and a gated water-control structure, and involve 52,000,000 cu yd of excavation and 1,800,000 cu yd of concrete.   The construction of these structures would involve only conventional methods.

Flood-Control Structures.—The flood-control structures consist principally of earth dikes and dams, constructed mainly with excavation spoil.   Construction of the flood-control structures except as a method of spoil utilization is conventional.

Housing and Facilities for Construction Workers.—The Canal Zone is a government-administered area in which, with rare exceptions, there is no private industry, business, or ownership of land or buildings.   Consequently,

TABLE 34.—EXCAVATION CLASSIFICATION; CANAL ZONE ROCK UNITS

Formation	Description	Unit weight (lb per cu ft)	Quantity (Thousands of Cu Yd)		Dynamite (lb per cu yd), dry excavation	Dynamite (lb per cu yd), wet excavation
			Dry	Wet		
(a) COMMON EXCAVATION						
Overburden and muck	Muck, silts, sands, clays, gravels, etc.	Variable	108,439	165,044	0	0
(b) SOFT ROCK						
Cucaracha	Largely dense greenish-gray clay shales highly slickensided within certain horizons. Black carbonaceous shales, sandstones, and conglomerates in subordinate proportions	135–140	125,584	27,406	0.30 (40%)	0.65 (40%)
Culebra	Medium-hard sandstones, soft sandy and carbonaceous shales	140±	146,647	8,216	0.30 (40%)	0.65 (40%)
La Boca	Dense silty or sandy dark-gray shales with intercalated sandstone beds sporadically present	140±	17,993	3,572	0.30 (40%)	0.65 (40%)
Gatun	Fine-grained argillaceous and calcareous sandstones with interbedded dense tuffs and conglomerates	120–125	14,330	7,409	0.30 (40%)	0.65 (40%)
(c) MEDIUM ROCK						
Caimito	Coarsely bedded medium-grained and fine-grained medium-hard limy sandstones and tuffs	130±	94,497	47,236	0.50 (40%)	1.00 (60%)
Las Cascadas	Agglomeratic tuff and tuff-breccia consisting of angular fragments of hard dark-gray andesite in a clayey dark-gray to light-green altered tuff matrix	140±	13,778	0	0.50 (40%)	1.00 (60%)
Bohio conglomerate	Subangular to rounded pebbles, cobbles, and boulders up to 2 ft in diameter in a dark-gray or brown generally coarse friable tuffaceous sand matrix	145±	81,722	50,463	0.50 (40%)	1.00 (60%)
(d) HARD ROCK						
Pedro Miguel and Bas Obispo agglomerate	Hard light-gray to dark-gray fine-textured to coarse-textured agglomerates and tuffs	155±	64,645	1,884	0.75 (60%)	1.50 (60%)
Basalt	Hard columnar-jointed basalt flows and intrusives	160–170	82,288	7,516	0.75 (60%)	1.50 (60%)
Total	....	....	749,923	318,746	....	....

housing, water supply, sewage disposal, power, communications, hospital and health facilities, schools, fire and police protection, and other facilities would have to be provided for the expanded population of the Canal Zone during canal construction. Housing and other facilities would be largely temporary in character but certain permanent construction would be performed to replace existing housing and other facilities which date back to the early 1900's and which are, or soon will be, obsolete.

PROGRAM

Various programs of construction were evaluated, and it was determined that a 10-year construction program to convert the existing canal to sea level would be the most favorable. A shorter program would result in less economical use of special plant for dry and wet excavation and would require more housing. A longer program would probably result in somewhat lesser costs, but would delay the fulfilment of the requirements of national defense.
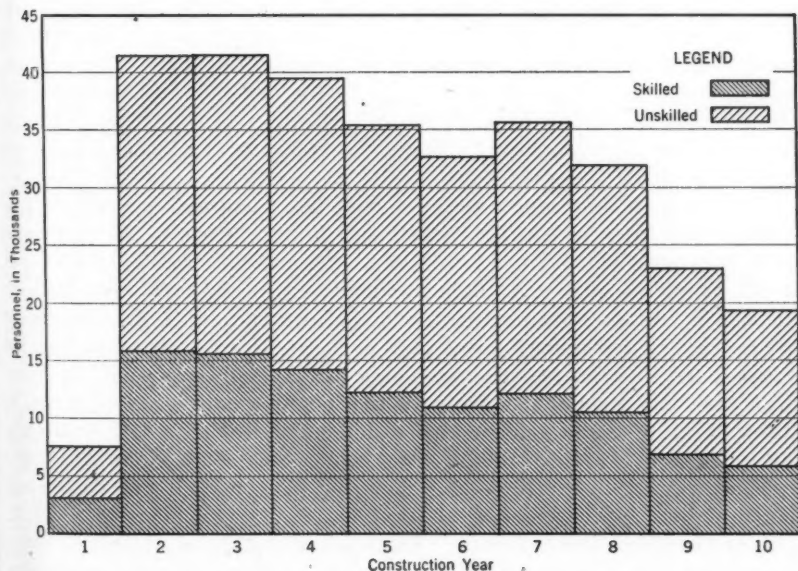
FIG. 94.—TOTAL PERSONNEL REQUIREMENTS, PANAMA SEA-LEVEL CANAL

PERSONNEL

Skilled and technical personnel would be obtained from the United States. The unskilled employees would be largely indigenous to the Caribbean area. Contracts with skilled and unskilled labor would include payment of transportation from the place of recruitment and return. Transportation of government workers' families and household furnishings would be provided. The labor requirements are shown in Fig. 94.

MATERIALS

Satisfactory concrete aggregates may be obtained from alluvial deposits of sand and gravel in the Chagres River or from basaltic rocks from the channel excavation. Local sources for cement, steel, and lumber are not adequate, and these items would be obtained from the United States. The large volumes of excavation throughout the entire length of canal and water transportation make available adequate and economical embankment materials for earth dam construction.

CONVERSION TO SEA-LEVEL CANAL BY STAGE DREDGING

The method of converting the present canal to sea level has a direct effect on the method of excavation; and, in turn, the various possibilities of performing the excavation have a direct bearing on the selection of the method of conversion.

In all previous studies of plans for effecting the conversion of the present canal, it was contemplated that Gatun Lake would be lowered by stages. In one study, seven stages of lowering were planned; in others, three stages were selected. Stage lowering would require progressive alteration of the existing locks so that traffic could be accepted at all stages of the lake. This program would involve certain risks to shipping and to the canal if auxiliary conversion locks were not built, since only one lane of locks would be open to traffic at a time while the other was being modified to accept traffic at the next lower stage.

In the current studies it was found that stage conversion could best be effected by lowering the lake in three stages—from El. 85 to El. 54 (the elevation of Miraflores Lake), from El. 54 to El. 22, and from El. 22 to sea level. For this plan of conversion, equipment would be required capable of dredging to depths of 72 ft, a not uncommon depth of excavation. It was concluded that the risk in depending on one lane of locks during conversion was not warranted and that continuous operation of two lanes of locks would be essential. Consideration was first given to the use of a third lane of locks, as planned in earlier studies, constructed to full summit height (El. 85) to overcome this serious handicap to the stage conversion plan. However, it was determined later that the three-stage lowering plan made it practicable and economical to construct the special two-lane, single-lift conversion locks, to El. 54, at Miraflores and at Gatun. The conversion locks would have a lower lift and would be no more costly than would the alternate plan involving a third single-lane, high-lift lock at each end of the existing lock sites and the conversion of the existing locks. The conversion locks would have upper sills at a low elevation to permit the transit of ships at each successive stage of lowering below El. 54. In this plan the existing locks would be abandoned when Gatun Lake was lowered to El. 54, and the temporary conversion locks at Gatun and Miraflores would then be placed in operation.

The disadvantages of the stage conversion plan are:

a. Conversion locks would be required and would add considerably to the cost of the project.

b. All dry excavation, approximately 750,000,000 cu yd of the total of more than 1,000,000,000 cu yd, would have to be excavated prior to the initial lowering of Gatun Lake (thus requiring a minimum construction program of 15 years to permit sufficient time for the two remaining stages of dredging).

c. Traffic in the canal would be interrupted during the drawdown for each stage.

d. Part of the material dredged during the second stage and all the material dredged during the final stage would require locking to sea for disposal, because the drawdown would prevent using Gatun Lake as a spoil area, and use of the locks for this purpose would interfere with canal traffic. (Approximately 55,000,000 cu yd would have to be transported to sea.)

e. The lowering of Gatun Lake for the final stages would deplete the storage area to such an extent that pumping of lockage water would be required.

SINGLE-STAGE CONVERSION BY DEEP DREDGING

A proposal by E. E. Abbott for conversion by deep dredging led to an investigation of the possibilities of adapting or developing dredging equipment so that all wet excavation could be performed in advance of the lowering of

the summit lake. This investigation indicated that such a method would be entirely feasible. Drilling, blasting, and excavating to depths of 145 ft below the surface of Gatun Lake would be required to provide the design depth in a sea-level canal.

In this plan, the present operating levels of Gatun Lake (El. 85) and Miraflores Lake (El. 54) would be maintained during the entire period of excavation of the canal. The level of Gatun Lake would be dropped to El. 81 a short time prior to final lowering to permit placing the flood-control system of the canal in operation. The canal would then be closed to traffic for about 7 days, during which

PERSPECTIVE

CROSS-SECTION

FIG. 95.—GATUN CONVERSION PLUG

the lakes would be lowered to sea level and the channel would be cleared of the barriers used to retain Gatun and Miraflores lakes during excavation.

The barriers or channel plugs used to retain the lakes would consist of the following: (1) A natural rock barrier across the sea-level channel on the north shore of Gatun Lake (Fig. 95); (2) a natural rock barrier in the sea-level channel separating Gatun and Miraflores lakes near the existing Pedro Miguel Locks (Fig. 96); and (3) a temporary steel dam in the north approach to the tidal
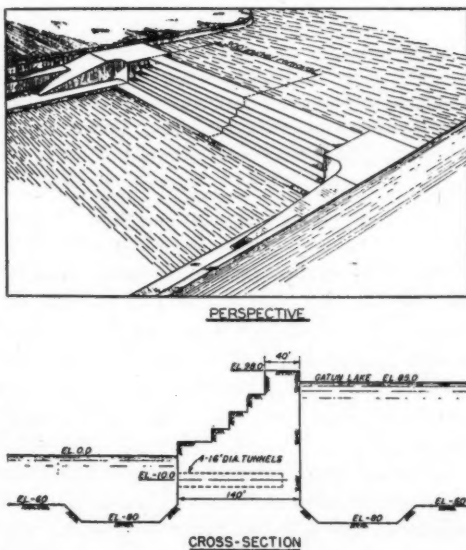
lock (Fig. 97). The natural rock plugs at Gatun and Pedro Miguel would be removed progressively by blasting and dredging during the 7 days that Gatun Lake was being lowered. The temporary steel dam in the approach to the tidal lock would be removed in sections by derricks when the water in the canal reached sea level.
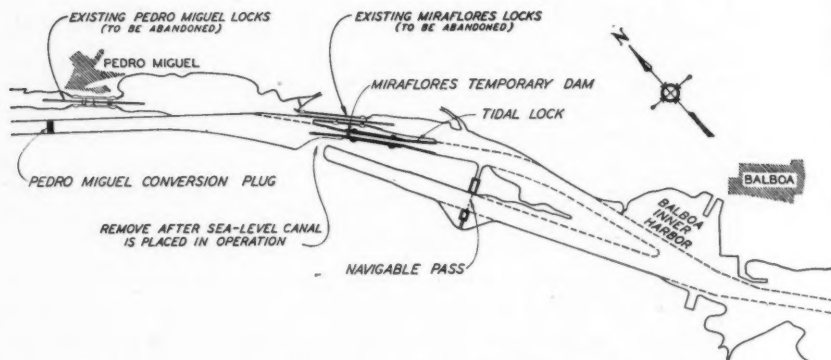

FIG. 96.—LOCATION PLAN, PEDRO MIGUEL PLUG AND MIRAFLORES TEMPORARY DAM

Gatun Conversion Plug.—The Gatun conversion plug would be of sandstone (Fig. 95). The bedding of this formation is massive, and the infrequent near-vertical joint planes are tight. The lake approach to the Gatun plug would be excavated by deep-dredging methods, but the immediate face of the plug would be carefully formed by line drilling and light blasting. The Atlantic approach would be excavated by dry methods. Four discharge outlets, 16 ft in diameter, would be tunneled in the plug from the downstream side of the plug to within about 20 ft of the face. The entire mass of the rock plug would be prepared for blasting and final removal by drilling vertical holes in a 5-ft pattern down to El. −50. Initially, the lake would be lowered through the flood-control spillways and the lock culverts. Later the tunnel outlets in the rock plug would be opened by blasting, using drill holes from the top of the plug for loading and firing, and finally the entire remaining mass of the plug would be blasted.

Pedro Miguel Conversion Plug.—The Pedro Miguel conversion plug would be generally similar to that at Gatun. It would be located near the upper approach to the existing Pedro Miguel Locks (Fig. 96). It would be placed as close to Miraflores Lake as sound rock permits, to reduce the volume of channel excavation required to be transported through the Miraflores Locks for disposal at sea. The agglomerate of which this plug is composed is a relatively impervious rock massively jointed. Any leakage through material of this type would occur along joints or fracture planes, and the condition would become evident during the progress of excavation. Grouting and sealing of the upstream side would be undertaken if found necessary. During the lowering of the lake, the plug would be removed progressively in a manner similar to that used at Gatun.

Miraflores Temporary Dam.—The upstream arm of the construction coffer-dam, enclosing the tidal lock, was considered for use as the plug for Miraflores Lake. The removal of the cofferdam and the excavation of the channel under the cofferdam that could not be done until drawdown of the lake had been completed would not be accomplished as rapidly as the removal of the Gatun and Miraflores plugs, and traffic would be suspended for a longer period. Therefore, a temporary steel dam is planned in the north approach to the tidal lock, illustrated in Fig. 97, which would be capable of holding Miraflores
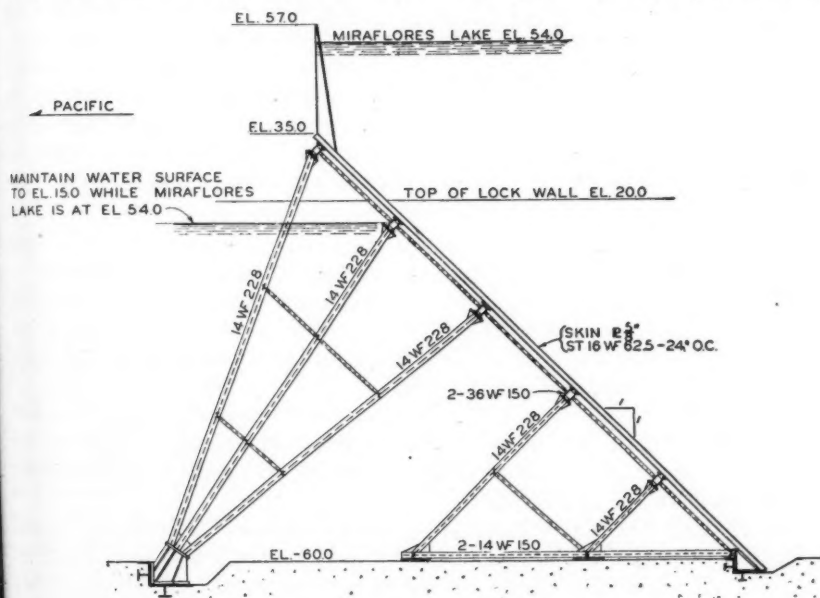

FIG. 97.—CROSS SECTION, TEMPORARY STEEL DAM IN TIDAL LOCK

Lake at its present elevation. It would be constructed in the completed lock approach above the upper gates prior to the removal of the construction coffer-dam. Short sections of earth embankments extending to high ground, with temporary concrete-retaining abutments on the lock walls, would be required to complete the closure. The dam would be constructed in a manner permit-ting rapid disconnection and removal of individual sections in 1 day by two derricks mounted on the lock walls.

Advantages of Deep-Dredging Plan.—The principal advantage of the deep-dredging method of excavation over the stage-dredging plan is in the saving of lock conversion costs. Also, the deep-dredging plan permits full use of the Gatun Lake area for deposit of all excavation spoil, with corresponding saving in excavation costs. Using Gatun Lake for the hauling of excavated materials would make it possible to construct the Caño Quebrado, Trinidad, and Monte Lirio flood-control dams with greater facility and at less cost. This plan would also result in less interference with canal traffic and would save about 5 years in construction time.

The net saving of the deep-dredging plan over the stage-dredging plan is estimated at $130,000,000.

DRY EXCAVATION

Conversion of the present Panama lock canal to a sea-level canal would require the excavation in the dry of 750,000,000 cu yd of material, practically all being between miles 21 and 24 and miles 27 and 36, Fig. 93. Considering the character and the volume of material to be removed, the nature of the terrain in this area, and the accessibility of spoil areas, it is evident that various types of equipment could be employed in the excavation and removal of material.

Operation Phases.—The most suitable method of excavation is that which has the widest general application and which results in the least cost for removal and disposal of the excavated material. It can be selected only by an examination of the principal phases of the excavation operations, namely: (*a*) Preparation of the material for excavation; (*b*) excavation; and (*c*) haul of excavated material to disposal areas.

(*a*) *Preparation of the Material for Excavation.*—For common material no preparation is required, and for soft rock preparation may or may not be required, depending on the selection of excavation equipment. For medium and hard rocks, preparation requires systematic drilling and blasting, the extent of which is determined by the character of the material and the type and and size of excavation equipment selected.

(*b*) *Excavation.*—A wide choice of equipment of various types and sizes is available. The utility of the different types of equipment depends on the character of material, the nature of its occurrence, and other factors discussed more fully elsewhere in this paper.

(*c*) *Haul of Excavated Material to Disposal Areas.*—This factor, because of the large quantity of materials involved, is of major, if not controlling, importance in the selection of the method of dry excavation. Accordingly, the methods of haul are discussed first.

Haul.—Haul refers to the transportation of the material from the place where it is excavated to the point of disposal—by truck, railroad, belt conveyer, scraper, scow, or other methods. The selection of the method is determined by the character of the excavation and its location in relation to the disposal area, the gradient between the two points, the location for suitable haul roads or conveyer lines—and in the case of scow haul by the availability of water transportation between the point of excavation and the disposal area.

Because the alinement of the proposed Panama sea-level canal takes advantage of the low terrain, the available dry spoil areas in the reach from Gatun Lake to Miraflores Lake are higher than the excavation, and are at an average distance of more than 5,000 ft from the center line of the canal. The bottom of the cut through this section is at approximately El. −70 and the average elevation of the top of the spoil areas would be between 400 ft and 500 ft. The distance and necessary gradient to overcome the difference in elevation would necessitate approximately 2 miles of truck haul on an upgrade. Truck haul, therefore, would not be an economical operation for most of the material.

However, a small part of the excavation (that above El. 270) could be hauled economically by trucks because the difference between that elevation and the spoil areas would be sufficiently small to permit removal with an average haul of about 1 mile.   In some locations, truck haul could be used economically below El. 270.   The exact elevation would be dependent on the location and elevation of the dry spoil areas as well as on the haul distance to the wet spoil area.

The difference in elevations between the bottom of the cut and the disposal areas would require approximately 5 miles of railroad on a 2% grade for the entire distance.   Removal by this method, therefore, was not considered feasible or economical.

The large amount of rainfall in this area, combined with the adhesive or sticky quality of the shale which forms a large part of the excavation, and the necessity for breaking the material into small sizes do not favor the use of belt conveyers.   In some sections of the canal excavation, this method might be used successfully but it does not appear feasible or economical for the operation as a whole.

The lack of suitable spoil areas within reasonable overland haul distances led to the consideration of a method of disposal in areas in Gatun Lake remote from existing and future channels.   Actually, this use of Gatun Lake is not new since the lake has been used for wet disposal of excavated materials since it was first created.   The principal new problem which this method of disposal presents, therefore, is that of effecting the transfer of dry excavated materials to suitable haulage equipment and their transportation to the dumping areas. These areas could be made accessible to railroad haul, averaging 15 miles, but trestles would have to be constructed in Gatun Lake.   Furthermore, it was found that the material could be hauled more economically in scows.   The Gatun Lake spoil areas are readily accessible to bottom dump scows and have capacity for more than 3,000,000,000 cu yd of spoil material below El. 65. Accordingly, a method of dry excavation was developed so that the material could be handled directly into scows to take advantage of the low cost water transportation.   The haul distance from point of excavation to point of disposal would vary from 1 mile to 23 miles and would average about 15 miles. The average cost of disposal by truck haul would be approximately twice as much as by scow haul.

Practically all disposal of dry excavation could be handled by the scow haul method by using a system of auxiliary barge channels, which would lie within the limits of the excavation area and would roughly parallel the center line of the new channel for the entire length of the dry excavation (Figs. 98 and 99). The initial auxiliary barge channels would enter the sea-level canal channel from the lake through natural inlets, or in some instances by way of an inlet or a channel excavated to connect with the present canal.   In constructing the barge channels, a pilot cut 180 ft wide to El. 90 would be made first along the length of the cut, followed immediately by a channel 110 ft wide excavated to El. 65, giving the channel a depth of 20 ft with the lake surface at El. 85. Material adjacent to both sides of the barge channel would be excavated to El. 90 casting directly, or recasting, into bottom dump scows in the barge
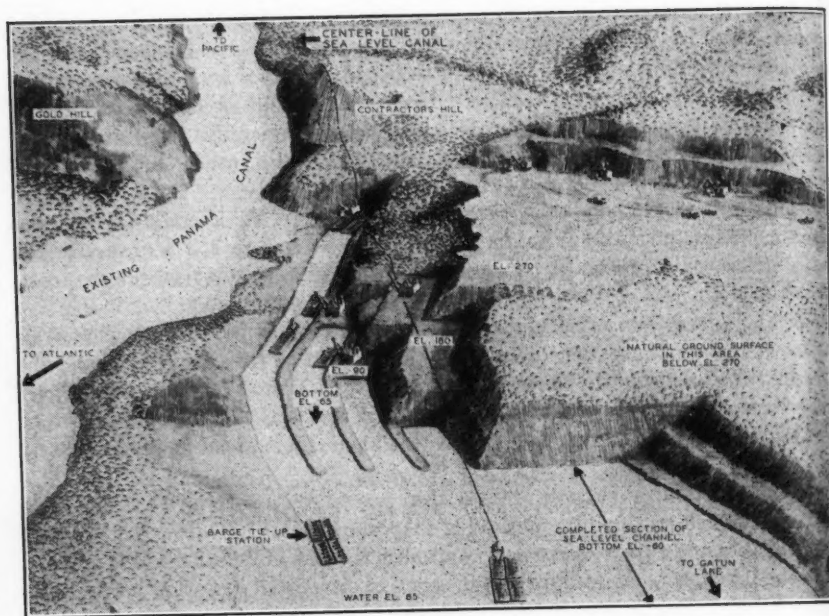
FIG. 98.—EXCAVATION ABOVE EL. 90, APPROACHING CONTRACTORS HILL FROM THE NORTH
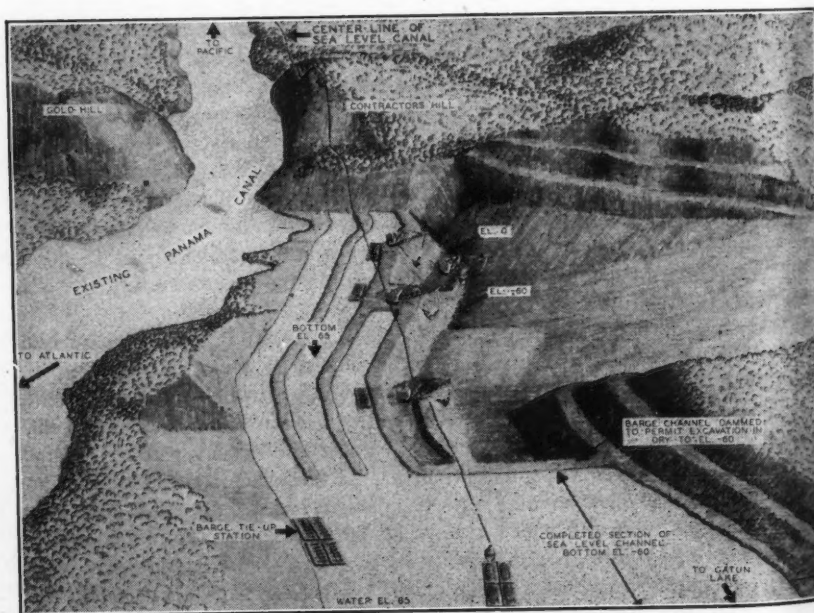
FIG. 99.—EXCAVATION BELOW EL. 90, APPROACHING CONTRACTORS HILL FROM THE NORTH

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channel. When a width of 180 ft had been excavated on either side of this channel, another barge channel would be excavated parallel to it, leaving a 70-ft berm between the two channels as shown in Fig. 98. This process would be continued until the excavation to El. 90 had been extended laterally to intersect the design side slopes at this elevation. All the material above El. 90 could be loaded or cast directly into the scows except that for the excavation of the very flat slopes, which would require recasting or trucking of the material to the scows.

For the excavation of material below El. 90 to project depth (Fig. 99), the barge channels would be blocked and dewatered progressively from one side of the channel to the other, each being excavated in turn to project depth. After drilling and blasting, the material in each dewatered channel would be removed by a dragline operating from the 70-ft berm separating the dewatered channel from the adjoining open or wet channel. Material below El. 90 would be excavated in two steps. In the first step, when the material was excavated to El. 0, the draglines would pick up the materials and load directly into the scows. In the second step, below El. 0, the materials would be cast on a bench and then picked up by draglines and loaded into scows.

The scow haul method of disposal facilitates the construction of the broad flood-control dams in Gatun Lake. This method would materially reduce the construction costs of these dams because they could be built to El. 65 with the 2,000-cu-yd bottom dump scows with very little extra haul distance. The dams could then be raised by using smaller barges, which would permit dumping to El. 75. The only important cost for the construction of the dams would be the special handling of a small part of the material from El. 75 to the required height at El. 82.

The scow haul method of disposal of all material excavated below El. 270 was adopted for construction planning purposes because it was the most economical method. The dry spoil areas are favorably located for the disposal of material excavated above El. 270 and the use of 20-cu-yd trucks was adopted for planning purposes. Additional economies may be realized by further study of truck haul for material below El. 270 in some locations.

Excavating Equipment.—The size and type of the hauling unit and the type of material to be excavated are the principal factors in selecting the size of shovel to be used. The capacity of the hauling unit should be equal to one or more times the load handled at one time by the excavator. For the material hauled by truck, studies have indicated that the 5-cu-yd shovel would be the most economical type of equipment for leading 20-cu-yd trucks.

Large excavating equipment, shovels and draglines with capacities greater than 25 cu yd, has proved economical in strip-mining operations where there is no problem of transportation. However, no records are available, involving the use of these large shovels or draglines, where the excavated material had to be transported. The scow haul method, previously described, using auxiliary channels, would lend itself readily to the use of this large equipment. This equipment has a wide operating radius, which permits it to make a cut about 180 ft wide. The large swinging radius of this equipment would facilitate the direct loading of scows reaching the equipment through barge channels.

This equipment could work high faces of 90 ft and the size of the dippers would require less drilling and blasting of the hard material. Also, the rugged construction of the large scows would withstand the loading shock of the material deposited by the large dippers. Furthermore, the large equipment would require less operating personnel for the same volume of material excavated than would smaller excavation equipment.

The lack of mobility of the large equipment would not be a serious disadvantage on this job because the excavation is highly concentrated. Large quantities of material would have to be removed for each position of each piece of equipment, because of the great depths and widths of cut. For the foregoing reasons, the large excavating equipment, having buckets of 25-cu-yd capacity or greater, was adopted for construction planning purposes for the excavation of practically all the material below El. 270.

Drilling and Blasting.—The methods of excavating and of hauling each have a definite bearing on the type and quantity of drilling and blasting required. The character of the material to be blasted determines the selection of the type of drilling equipment. The spacing of holes and the amount of blasting, expressed in terms of pounds of powder per cubic yard, are determined by the size of the hauling, loading, and drilling equipment.

The classification of the material to be drilled and blasted is shown in Table 34. From studies of drilling performed by contractors and The Panama Canal in this area, it was determined that the rotary drill would be the most suitable and economical for soft and medium rocks; and the percussion or churn drill for hard rock.

Excavation with small shovels would be on rather low faces, about 30 ft high, and would require fairly close spacing of drill holes varying from 8 ft by 8 ft for the hard rock, drilled by percussion-type wagon drills, to 18 ft by 18 ft for for the soft rock, which would be drilled with rotary drills.

Most of the dry excavation, 700,000,000 cu yd out of 750,000,000 cu yd, would be handled with the large shovels and draglines, 25 cu yd and more, and hauled in large scows. The digging height of the large shovels is about 90 ft, and therefore depths of drilling of 90 ft. would be practicable. The size of the dippers and the fact that the scows could accept pieces as large as those handled by the dipper, would permit 30-ft by 30-ft spacing of the drill holes in soft and medium rocks, materially reducing the cost of drilling. For the hard rock, drilling could be done to the full depth of the 90-ft faces, but coyote or tunnel blasting would be employed to effect savings in drilling and in blasting.

The weight of powder required would vary with the material, size of blast, and local conditions. Past experience in the Canal Zone indicates that the powder requirements will vary between $\frac{1}{4}$ lb per cu yd for the soft rocks to $\frac{3}{4}$ lb per cu yd for the hard rocks, when the drilling and blasting are done in the dry.

WET EXCAVATION

The deep-dredging scheme for conversion of the Panama lock canal to sea level requires the development of special equipment for excavation to the unprecedented depth of 145 ft below the existing level of Gatun Lake. It also calls for subaqueous drilling and blasting to far greater depths than have pre-

viously been required in wet excavation. Since these two considerations are controlling factors in setting up the construction procedure for wet excavation, they will be discussed first, followed by a brief discussion of proposed construction procedure.

Dredge Design Contracts.—To obtain the views and advice of the dredging industry as a whole regarding the feasibility of dredging to depths of 145 ft, a preliminary conference was held in Philadelphia in March, 1946, attended by representatives of thirty dredging contractors, dredge designers and builders, the Corps of Engineers, and The Panama Canal. At this conference, problems involved in construction of a sea-level canal were discussed, and it was the consensus that deep dredging was practicable. Subsequently, contracts were made with dredge manufacturers for design of the following types of dredges:

Type	Company	Maximum dredging depth (ft)
Hydraulic	Panama Contractors	145
Bucket ladder	Cuba Manufacturing Company	145
Dipper	Bucyrus-Erie Company	85

Revised specifications of the bucket-ladder dredge show that, with the ladder at a 45° inclination with the horizontal, the maximum dredging depth would be 148 ft.

Panama Contractors, organized especially for the hydraulic dredge contract, is a combination of the Atlantic Gulf and Pacific Company, Standard Dredging Corporation, and Gahagan Construction Company.

The maximum dredging depth of 148 ft specified for the hydraulic and bucket-ladder dredges is that required for excavation to grade, working from the normal Gatun Lake elevation of 85 ft. The maximum dredging depth of 85 ft for the dipper dredge is presently considered to be the greatest depth to which a dredge of this type can economically excavate. It is proposed, therefore, that dipper dredges will excavate rock to an 85-ft depth, followed by deep-digging bucket-ladder dredges, excavating to grade. Bucket-ladder dredges, capable of dredging to a maximum depth of 90 ft, would also be utilized for initial rock excavation. The hydraulic dredge would be used only for the excavation of common material and soft rock, such as the Gatun sandstone. Development designs and cost estimates for the construction of the three types of dredges have been completed.

Hydraulic Dredge.—In present hydraulic dredge design, the main pump of the dredge is located near the water surface. Atmospheric pressure, in forcing the dredged materials to the pump from depths of 40 ft or 50 ft, is sufficient to supply ample velocity head, overcome friction losses, and support a column of mixture having a density substantially greater than that of water. As deeper dredging is undertaken, the entrance loss and velocity head remain the same, but the friction head and the pressure necessary to support the columns of mixture increase. Various schemes to bring a richer mixture to the dredge pump were considered, such as utilizing nozzles in the suction, depressing the elevation of the main pump, placing a propeller-type pump in the suction, or placing a booster pump on the dredge ladder below the water surface. It was
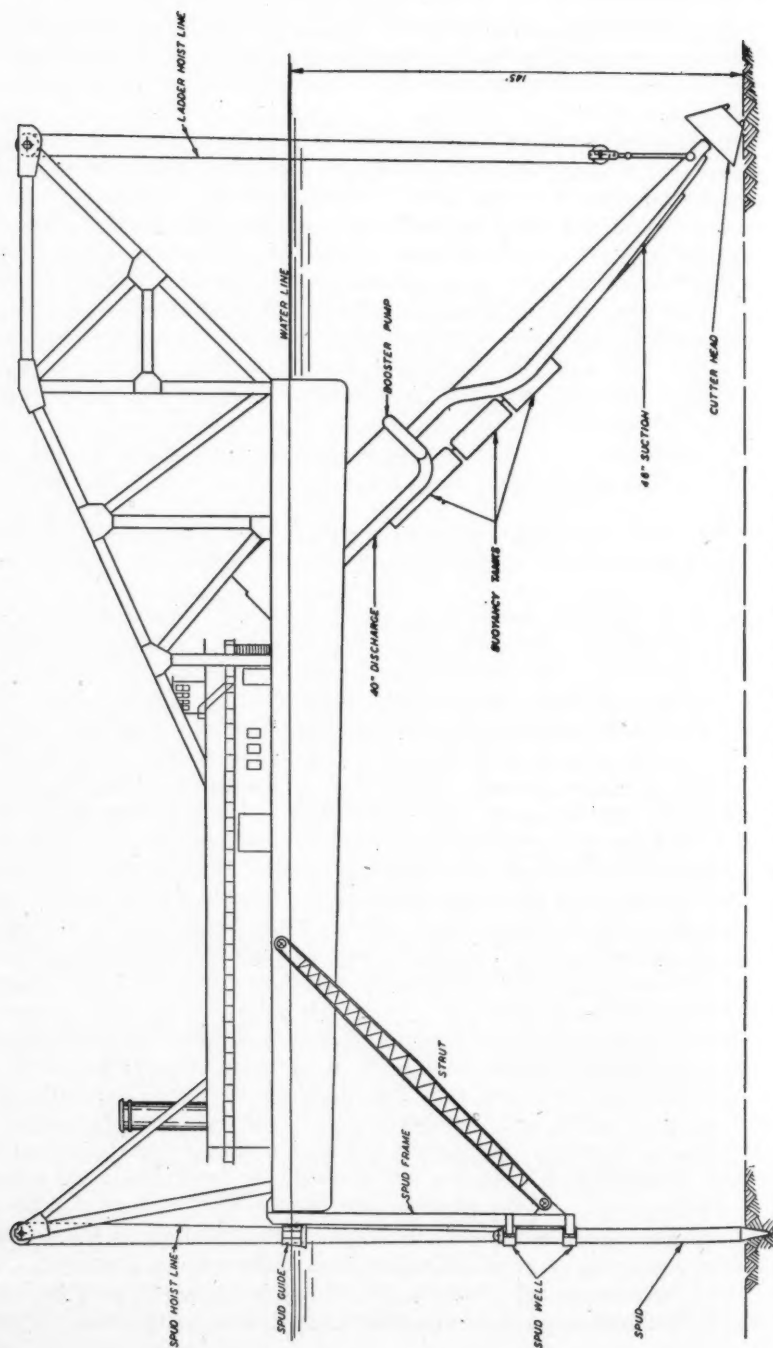
FIG. 100.—PROPOSED 40-IN. HYDRAULIC DREDGE

decided that the last scheme was the most practicable of those considered. Fig. 100 shows a general arrangement of the ladder and booster pump.

Two basic mechanical problems that required consideration were: First, the design of a dredge ladder approximately 200 ft long and capable of supporting the booster pump and attendant machinery; and, second, the design of spuds and a spud frame to hold the dredge in place. Both designs were developed sufficiently to indicate that they are entirely practical.

The deep-dredging ladder is provided with buoyancy tanks (Fig. 100) for assistance in hoisting and for better stress distribution. The booster pump is enclosed in a cofferdam to permit removal of obstructions from the booster pump when the ladder is not completely raised.

For holding the dredge in place for deep dredging, a frame extending 90 ft below the water surface is used to hold the spuds. This frame, which is braced to the hull, would be replaced by shorter frames when dredging to more conventional depths. The frame would consist of two main vertical members of sufficient size and section to serve as wells for the spuds which would extend below the wells to a depth of some 160 ft. Fig. 100 shows a schematic layout of this arrangement. General features of the hydraulic dredge are as follows:

Feature	Description
Suction	46-in. diameter
Discharge	40-in. diameter
Hull	265 ft long, 60-ft beam, and 20 ft deep
Power: Main pump	Steam turbine, reduction gears, 8,000 hp normal, and 10,000 hp maximum
Auxiliaries	Electrically driven from turbogenerators
Cutter motor	Electrically driven, 2,000 hp to 2,500 hp
Booster	Electrically driven from shore, 8,000 hp

Bucket-Ladder Dredge.—The bucket-ladder dredge has been used quite extensively in Europe for many types of subaqueous excavation and is generally recognized as a European development. It was the most successful type of dredge used by the French at the Panama Canal, and under the American regime several of the old French bucket-ladder dredges were rebuilt and placed in service, one of these remaining in commission until 1920. Use of the bucket-ladder dredge in the Western Hemisphere, the Dutch East Indies, the Federated Malay States, and Russia has been confined principally to placer mining for gold, platinum, and tin. The bucket-ladder dredge would be used on the Panama Canal project for excavation of blasted rock, work which has been performed successfully by such equipment but which is more unusual than the excavation of gravel and conglomerate.

The bucket-ladder dredge has certain distinct advantages over other types. For example, its operation involves a minimum of lost motion as compared with the dipper dredge; it does not require large quantities of water, as does the hydraulic dredge; it requires relatively little power; and it can dredge to great depths. One bucket-ladder dredge in the California gold fields is excavating to a depth of 124 ft, and 200-ft dredging depths have been predicted for the near future by leaders in the placer dredging industry.

Bucket-ladder dredges of two limiting digging depths are under consideration—one for dredging to a maximum depth of 90 ft and the other for dredging to a maximum depth of 145 ft. The general scheme of operation would be to utilize the 90-ft dredge for excavation to 90-ft depths, followed by deep-digging dredges for excavation between 90-ft and 145-ft depths. For excavation to 90 ft below water level, a dredge hull of shorter length could be utilized than for 145-ft digging; also the length of the digging ladder could be reduced from 239 ft to 163 ft. The original and operating costs of the 90-ft dredge would obviously be less than those of the 145-ft dredge. However, machinery for dredges of both sizes would be identical to permit interchangeability of parts.

The major item of cost involved in rock excavation in the wet is that of drilling and blasting, and, in general, unit costs decrease rapidly as the spacing between drill holes is increased. In the bucket-ladder dredge design, therefore, it was desired to provide the largest practicable buckets to permit maximum spacing of drill holes. Consideration was given to bucket sizes of 6 cu yd, 5 cu yd, 4 cu yd, 3 cu yd, and 2 cu yd. Representatives of the contractors and The Panama Canal agreed that the 2-cu-yd bucket is the maximum size that should be adopted, because of difficulties in casting very large buckets and because of the tremendous weight of the digging ladder and the bucket chain.

The 2-cu-yd bucket has three times the capacity of the largest buckets now in use in placer dredging. A few European dredges have been equipped with buckets having capacities up to 2 cu yd, but the working conditions that they encountered were not severe, the maximum dredging depth being only about 50 ft and the material excavated being sand, silt, shell, and clay. The bucket-ladder dredge *Corozal*, formerly owned by The Panama Canal, was equipped with 2-cu-yd buckets for soft digging and $1\frac{1}{2}$-cu-yd buckets for rock excavation.

Spoil disposal would be accomplished by scows, loaded by conveyers on the starboard and port sides of the dredge. Consideration was also given to a stern conveyer system composed of the necessary conveyer unit for final disposal of spoil ashore. However, detailed study indicated that the latter method of disposal is not desirable because of the very long conveyer lines required and because of difficulties in designing the spud arrangement. By elimination of the stern conveyer system, spud design is simplified. Fig. 101 shows an elevation of the proposed bucket-ladder dredge, including the proposed spud arrangement.

General features of the bucket-ladder dredge design for 145-ft dredging as developed under contract are as follows:

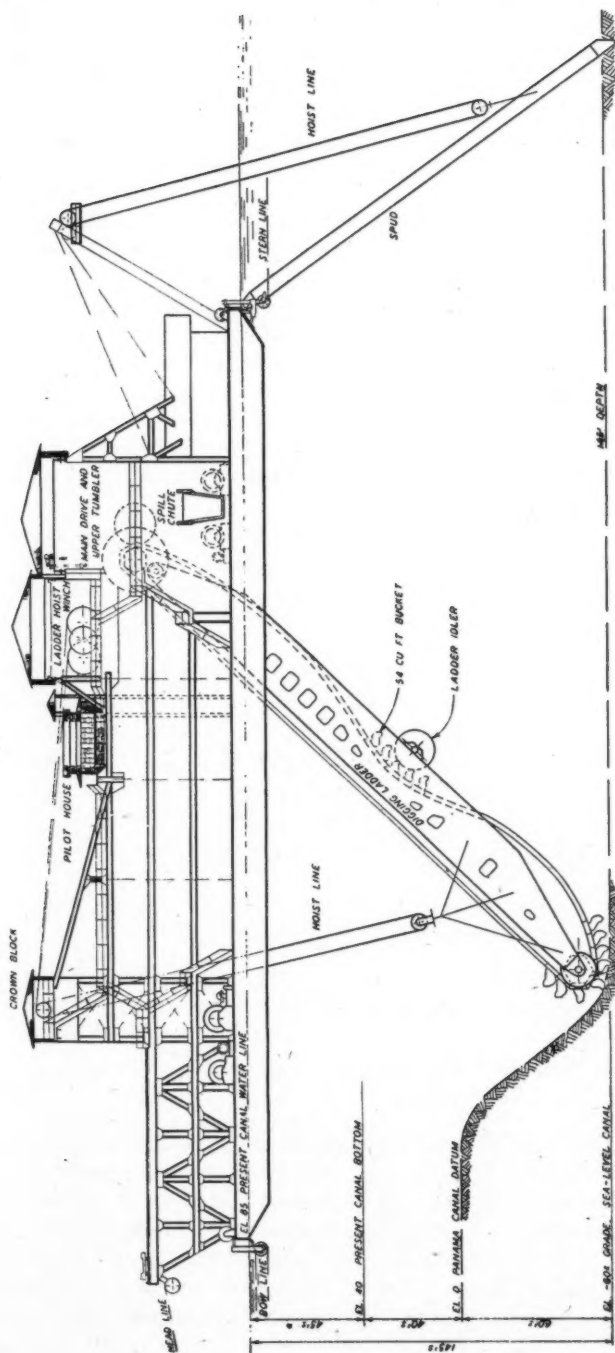| Description	Quantity
Dimensions of Hull, in Feet—	
Length	371
Beam	100
Depth	13
Bucket capacity, in cubic yards	2
Power; number of 1,600-hp, deisel electric engines with identical direct-connected generating equipment	4
Main drive, in horsepower	2,500
Total connected load, in horsepower	5,294
Length of digging ladder, in feet	239

Fig. 101.—Proposed 54-Cu-Ft Bucket-Ladder Dredge

Dipper Dredge.—The dipper dredge is a proved tool for rock excavation, its principal limiting factor being that of digging depth, since the time required for the digging cycle increases rapidly as depths increase. Although, under the deep-dredging plan, the dipper dredge contemplated could not excavate to final grade from the existing level of Gatun Lake, its maximum dredging depth of 85 ft is considerably in excess of that of any dipper dredges previously developed. Furthermore, large quantities of hard rock are located above the 85-ft depth in Gatun Lake (that is, above El. 0) within the economical digging range of the dipper dredge. After canal construction, the dipper dredge could be used effectively for maintenance.

The dipper dredge design incorporates a balanced hoist and a walking stern spud, which are shown diagrammatically in Fig. 102.

The balanced hoist consists of a movable counterweight connected to the main hoisting machinery by a double three-part hoist. The counterweight is lowered as the dipper is raised, or vice versa, effecting an approximate balance for the dead weight of the dipper and handle, so that practically the entire hoist motor power is available for digging. The movable counterweight is located at the stern.

The stern spud would be mounted as a trailing or walking spud in a well at the stern of the hull. For the purpose of controlling the angular movement and applying power to walk the spud or push the dredge ahead, a powerful hydraulic ram on the main deck would be pin-connected to a heavy frame surrounding the spud.

The dredge would be electrically operated from a 4,000-v, three-phase, 60-cycle alternating current generated aboard by four identical diesel engine generating sets. The combination of any three sets would provide sufficient power for full-speed operation of the dredge, and the fourth would be a stand-by unit. General features of the dipper dredge design are:

Description	Quantity
Length of Hull, in Feet—	
Length...	200
Beam...	85
Depth...	14
Dipper capacity, in cubic yards.......................	20 to 30
Power; diesel electric power from 1,000-hp engines (adaptable for shore power)...........................	....
Maximum digging depth, in feet.......................	85
Bail pull, in pounds (stalling).........................	500,000

Deep Drilling and Blasting for Wet Excavation.—Preparation of rock for excavation under the deep-dredging plan requires subaqueous drilling and blasting to depths far in excess of those ever before encountered in canal excavation. Therefore, it was considered necessary to determine the effects of these depths, as compared to conventional depths, on (1) drilling and blasting operations, including maneuvering of equipment and rate of drilling; (2) requirements of explosives; and (3) dimensions of the blasted rock.

PILOT HOUSE

20-30 CU YD DIPPER

FORWARD SPUD

WALKING STERN SPUD
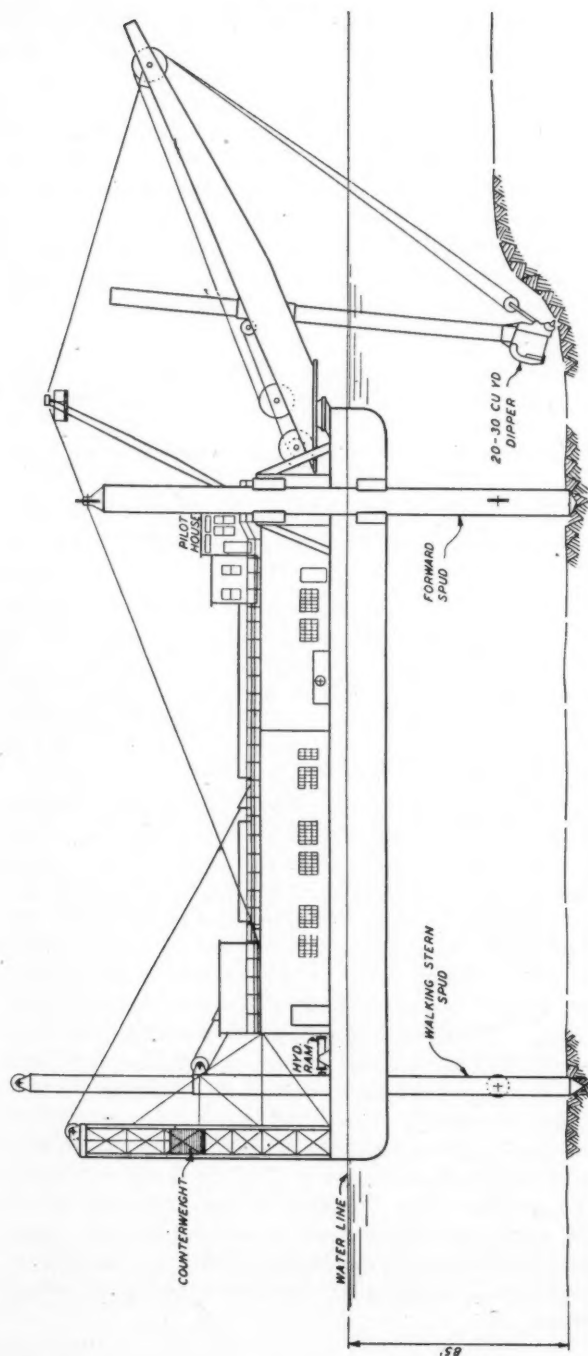
HYD RAM

COUNTERWEIGHT

WATER LINE

.58

FIG. 102.—Proposed 20-Cu-Yd to 30-Cu-Yd Dipper Dredge

To obtain the desired information, a deep-drilling and blasting test was conducted in which rock was drilled, blasted, and excavated in three lifts of approximately 25 ft each from 50 ft to 125 ft below water surface. An accurate record was kept of all drilling, loading, and blasting operations to determine drill performance, the difficulty and time required for maneuvering equipment, setting casing, and loading. Fragmentation of the excavated material was observed to ascertain the effect of hydrostatic head on breakage. The same quantity of explosive per cubic yard was used for each lift.

All drilling was performed by rotary drill, mounted on a barge with necessary winches for maneuvering. Excavation was accomplished by derrick barge equipped with a 3-cu-yd clamshell bucket. From the test, it was concluded that:

(1) Drilling operations with a properly designed drill boat are not appreciably affected by the depths of water that would be encountered in the deep-dredging plan.

(2) The effect of increased depth of water (within the limits of this test) is negligible.

(3) Dimensions of blasted rock obtained in all three lifts did not differ appreciably. The same unit explosive loading and hole spacing were used for each lift.

The rotary drill was selected because it could be utilized in the test without extensive and costly alterations, whereas other types of equipment would have required major reconversion with resulting delay of the test. Furthermore, sufficient data regarding drilling rates at conventional depths with the percussion and rotary types of drills were available to permit reasonably accurate conversion of deep-drilling results with one type of drill to those that might be expected with the other.

In the preparation of estimates for drilling, blasting, and dredging, rock has been divided into three classifications—soft, medium, and hard—on the basis of extensive core borings. Experience on the Panama Canal indicates that, for soft and medium rocks, subaqueous drilling rates are higher and costs are lower, using the rotary type of drill. For hard rock, drilling rates of the rotary and percussion drills are almost equal, or slightly favor the rotary type, but bit costs are higher for the rotary drill. The net result is that in hard rock the drilling cost per foot for the rotary drill may be somewhat higher than that for the percussion type. This latter factor, however, is not important because hard rock comprises only 5% of the total subaqueous drilling and blasting in the project, or 4% of the deep subaqueous drilling and blasting.

Wet Excavation Procedure.—Excavation would be performed in the dry throughout the Gatun Lake section to El. 90 (5 ft above normal lake level) and throughout the Miraflores Lake section to El. 60 (6 ft above normal Miraflores Lake level). Excavation below El. 90 in Gatun Lake and below El. 60 in Miraflores Lake, except for certain sections adaptable to dry excavation behind plugs, would be performed by dredges. Dredging conditions in both the Atlantic and Pacific sea approaches are suitable for the use of existing hydraulic and dipper dredges.

The deep-dredging plan would require that about 60% of the total dredging be performed to depths of 85 ft or less; and about 45% of the total dredging, to depths of 65 ft or less.   Therefore, a number of present-day dredges that can excavate to a depth of 65 ft with minor or no alterations and some dredges capable of excavating to 85-ft depths without extensive modification could be used on the project.

Plans contemplate the use of conventional hydraulic dredges for the initial excavation of common material to a depth of 65 ft, or possibly to a depth of 85 ft.   Common material from the bottom of the initial excavation to a depth of 145 ft would be removed by the 40-in. hydraulic dredges previously described. The number of passes or lifts that would be required in excavation to final grade would vary with materials and their ability to flow to the dredge suction without objectionable caving of the bank, but usually the excavation would be performed in about 30-ft lifts.   Hydraulic dredge spoil would be pumped to the nearest available disposal area, and for most of the material in Gatun Lake only a floating pipe line would be required.   Pipe-line lengths would vary from 3,000 ft to 12,000 ft and, when in excess of about 5,000 ft, shore-powered boosters would be introduced into the line.   Pipe lines would be laid to disposal areas on both sides of the canal so that there would be no interference with traffic. For example, a dredge working east of the center line would discharge to the east, and vice versa.   Under certain conditions, where it might be necessary to pump across the channel, submerged pipe lines would be used.

Following the removal of common material or overburden by hydraulic dredge, rock would be drilled and blasted for subsequent removal by dipper and bucket-ladder dredges.   Most, if not all, of the drilling and blasting would be performed by rotary drill boat.

Blastholes would be overdrilled the equivalent of one half of the hole spacing, which would vary from 10-ft centers in hard rock to 14-ft centers in soft rock.   It is anticipated that lifts of about 30 ft each would usually be blasted, but this estimate would vary considerably, depending on local conditions and type of excavating machinery utilized.   Future drilling and blasting tests may indicate the desirability of drilling and blasting to grade in one operation. Blastholes would be drilled approximately 6 in. in diameter, and for the larger blasts as much as 20,000 lb of dynamite would be fired in one shot.

Following drilling and blasting operations, rock would be excavated by dipper and bucket-ladder dredges, loaded into 2,000-cu-yd dump scows and towed to the dumping grounds by 1,500-hp tugs.   Spoil from the Gatun Lake area would be utilized in the construction of the flood-control dams across Gatun Lake, whereas that from Miraflores Lake and from the Atlantic and Pacific approaches would be deposited at sea.

SUMMARY

The methods presented in this paper are feasible and show a decided saving over other methods studied.   The estimate showed that approximately 15% could be saved on the cost of the dry excavation alone, whereas the use of the

large dry excavation equipment would reduce the required personnel by 9,000 workers, or by approximately 30%.   This saving would be reflected in the lowered cost of housing, utilities, services, and mobilization, making an over-all saving of between 10% and 12% for the project.   In addition to the cost saving, there would be a saving in the time required for construction.   Any reasonable schedule to perform the required excavation by stage dredging would cover a 15-year period, whereas the adopted plan could be scheduled for a 10-year construction period.